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Behavior of RC Eccentric Corner Beam-Column Joint under Cyclic Loading: An Experimental Work

A. M. Basha [a*], Sabry Fayed [b]

[a] *Assistant Professor, Department of Civil Engineering, Faculty of Engineering - Kafrelshiekh University, Kafrelshiekh, Egypt.*

[b] *Assistant Professor, Faculty of Engineering, Kafrelshiekh University, Kafrelshiekh, Egypt.*

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Abstract

The present research investigates experimentally the behavior of the reinforced concrete BCJ joint under quasi-static cyclic loads for different mount of the shear reinforcement. The specimen consisted of two columns and two beams; one is a free end and the other is a fixed end. The shear reinforcement of the joint was 1Ø6 mm, 2Ø6 mm, and 3Ø6 mm. Three specimens were tested under quasi-static cyclic loading up to failure. The cracking loads, ultimate loads, deflection of the free end of the loaded beam, crack patterns and failure modes for BCJ were recorded and analyzed at each cycle. Also energy dissipation and stiffness degradation of all specimens were discussed. The experimental result indicates that the increase in the amount of the stirrups of the joint transmitted the main failure into the column far away from the joint. The increase in the amount of joint stirrups enhanced the joint capacity. The stirrups are most effective in is the middle third part of the BCJ than others two parts.

Keywords: Beam-Column Joint; Reinforcement of the Joint; Failure Modes; Crack Pattern; Deflection Behavior; Quasi-Static Cyclic Loading.

1. Introduction

The Beam Column Joints (BCJ) are one of the most important structural elements in concrete structures. This is due to their construction difficulty and its complicated design. In several structures subjected to earthquake, the BCJ was the main reason of its collapse. Kusuhara and Shiohara [1] tested a ten half-scale reinforced concrete beam-column joint sub-assemblages loaded up to failure by statically cyclic loading simulating earthquake loading, to obtain fundamental data including stress in bars after yielding and joint deformation. The cross-section of the beam is 300 × 300 mm and that of the column is 300 × 300 mm in all the specimens. Three sets of hoops of Ø6 were placed in the beam-column joints in all the specimens; the ratio of joint shear reinforcement is 0.3%. It was found that the story shear capacity of the specimen with transverse beams, in which the damage of the joint was severe, was improved. Also, in case of damage of joints were severe, bond actions of beam bars passing through the joints kept lower level than the bond strength. Poor anchorage length of beam bars in exterior joints led lower story shear capacity, yielding of column bars and severe damage in the joint.

Leslie M. Megget [2] tested four external reinforced concrete beam-column sub-assemblages under pseudo seismic cyclic loading. Two different forms of beam bar anchorage were tested, the normal 90-degree “standard hook” and the continuous U-bar detail. It was found that the maximum beam elongations between 2.7 and 3.8% of the beam depth were measured in all the units tested with 500E Grade beam reinforcing, about 35% greater than those measured for the

* Corresponding author: ealibasha@eng.kfs.edu.eg

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same sized beams with Grade 430 reinforcing at the same level of ductility. Constantin and et al. [3] investigated the effectiveness of crossed inclined bars (X-bars) as joint shear reinforcement in exterior reinforced concrete beam–column joints under cyclic deformations. The results showed that the specimens with X-bars as the only joint shear reinforcement exhibited high values of load capacity in most of the loading cycles and increased hysteretic energy dissipation practically in the entire loading sequence.

Kaungand and Wong [4] studied the effectiveness of horizontal stirrups in the joint core for exterior beam-column joints under reversed cyclic-load with a non-seismic design according to British standard BS 8110. It was found that horizontal stirrups which were provided in beam-column joints with non-seismic design improve effectively the seismic behavior and enhance the joint shear strength.

Tarek El-shafiey and et al. [5] investigated experiments consisted of four beam-column joint specimens subjected to torsional moment acting on the beam. They studied the effect of joint stirrups. They shed the light on the importance of longitudinal side reinforcing steel configuration. It was found that the existing of joint stirrups and developed length of beam steel transferred the failure away from the joint panel. Recently, the use of high-strength reinforcing bars has increased in order to reduce the number of reinforcing bars. Hyeon-Jong Hwang and et al. [6] introduced an experimental study was performed to evaluate the seismic performance of beam-column joints using grade 600 MPa (87.0 ksi) bars for beam flexural reinforcement decreased by a maximum of 25% due to the increased bond-slip at the joints.

In eccentric beam-column joints, the axis of the spandrel beams is offset from the axis of column. When the eccentric joints subjected to earthquake loads, it was considered that additional shear forces produced by torsion moment from beams, severely act on the joints. Moreover, brittle shear failures of eccentric joints subjected to additional shear forces were observed from the previous earthquake damages. The effect of eccentricity on the degradation of shear strength, stiffness and deformation capacity of beam-column joints have been carried out by numerical analysis that it induced by Takashi Kashiwazaki and Hiroshi Noguchi [7]. The result indicates that the maximum story shear forces did not increase when the beam flexural yieldings have occurred.

The seismic performance of reinforced concrete beam-column joints was investigated experimentally by many researchers such as Minakshi Vaghani and et.al [8], A. Benavent-Climent and et.al [9] and A.M. Elsouri and Harajli [10]. All previous study included the following parameters: strength, displacement, ductility, energy dissipation capacity, drift reversals, reinforcement detailing requirements and stiffness. It was found gap in research of behavior of RC beam column joint in case of two beams; in plane and out of plane so, in the current research, the behavior of reinforced concrete corner eccentric beam column joint with different shear reinforcement ratio under cyclic loading was investigated experimentally.

2. Experimental Program

2.1. Specimens Details

The experimental program consisted of a three reinforced concrete (RC) eccentric beam-column corner joint specimens. A corner beam-column joint shown in Figure 1, was considered. It consisted of a half column connected to the top and bottom of the joint and part of two beams up to 50% of the span, which corresponded to the points of contra-flexure in beam and column under lateral loads. The joint was scaled to 1/3 size for experimental work. Symmetric boundary conditions were maintained at both ends of the column for isolation of a single unit of beam-column joints. In the present research, each specimen consisted of a column and two perpendicular beams. The first beam was loaded with reversible cyclic loads and the second beam did not loaded so it called the transverse beam. The axis of the loaded beam is offset from the axis of column by 50 mm.

The Egyptian code [11] was used in the design of specimens. All specimens have the same dimensions and reinforcement details. 3D isometric of specimens is shown in Figure 2. The column has a cross section of 100 × 200 mm and the total height of 1300 mm. the column was connected to two beams with 100 mm in width and 200 mm in thickness. The column was reinforced by 4Ø12 in the longitudinal direction. The transversal stirrups of the column were 12Ø 6/m except for the joint panel. The shear reinforcement of joint panel was a variable parameter as listed in Table 1. The ratio of shear reinforcement of the joint panel of specimen P_1 is 0.36% while that of 0.73% for P_2 and 1.09% for P_3 as shown in Table 1.For the loaded beam (A), the top and bottom main longitudinal bars were 2Ø 12 in longitudinal direction while 10Ø 6/m were used as vertical stirrups. The embedded length of lower and upper main steel of loaded beam was 245 mm to keep the bond actions of loaded beam bars passing through the BCJ small than the bond strength. For the transverse beam (B), the longitudinal main steel was 2Ø12 mm as top and bottom reinforcement. In addition 6Ø 6/m was used as vertical stirrups. Reinforcement details and concrete dimension of all specimens were shown in Figure 3 and it detailed in Table 1.

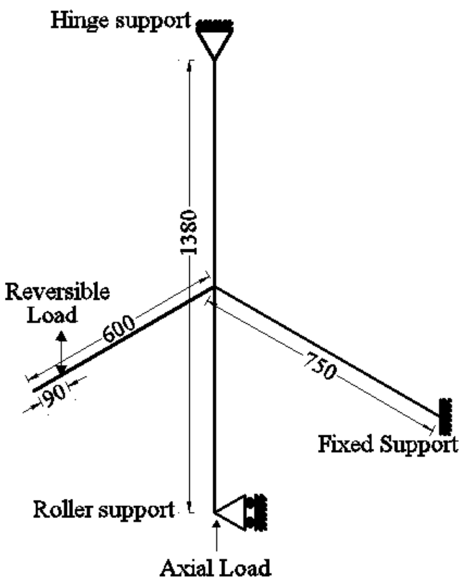


Figure 1. Isolated corner beam-column joint of all specimen

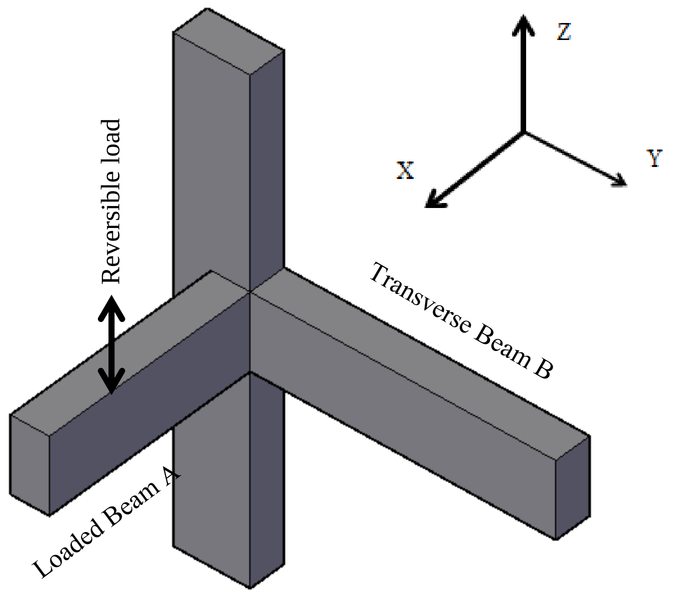


Figure 2. 3D isometric of the specimens

Table 1. Details of the tested specimens

	Beam A				Beam B				Column					Joit	
	Dimension		Reinforcement		Dimension		Reinforcement		Dimension			Reinforcement		Reinforcement	
Specimen	**Cross-secion.mm**	**Length mm**	**Top and bottom bars**	**stirrups**	**Cross sec.mm**	**Length mm**	**Top and bottom bars**	**stirrups**	**$b_c$mm**	**$t_c$mm**	**$h_c$mm**	**Longitudinal bars**	**stirrups**	**stirrup**	**Stirrup percentage %**
P_1														1Ø6mm	0.36
P_2	100x200	600	2 D 12 mm	D 6 @ 100 mm	100x200	750	2 D 12 mm	D 6 @ 94.3 mm	200	100	1300	4 D 12 mm	D 6 @ 84 mm	2Ø6mm	0.73
P_3														3Ø6mm	1.09

Table 2. Concrete mix per 1 m^3

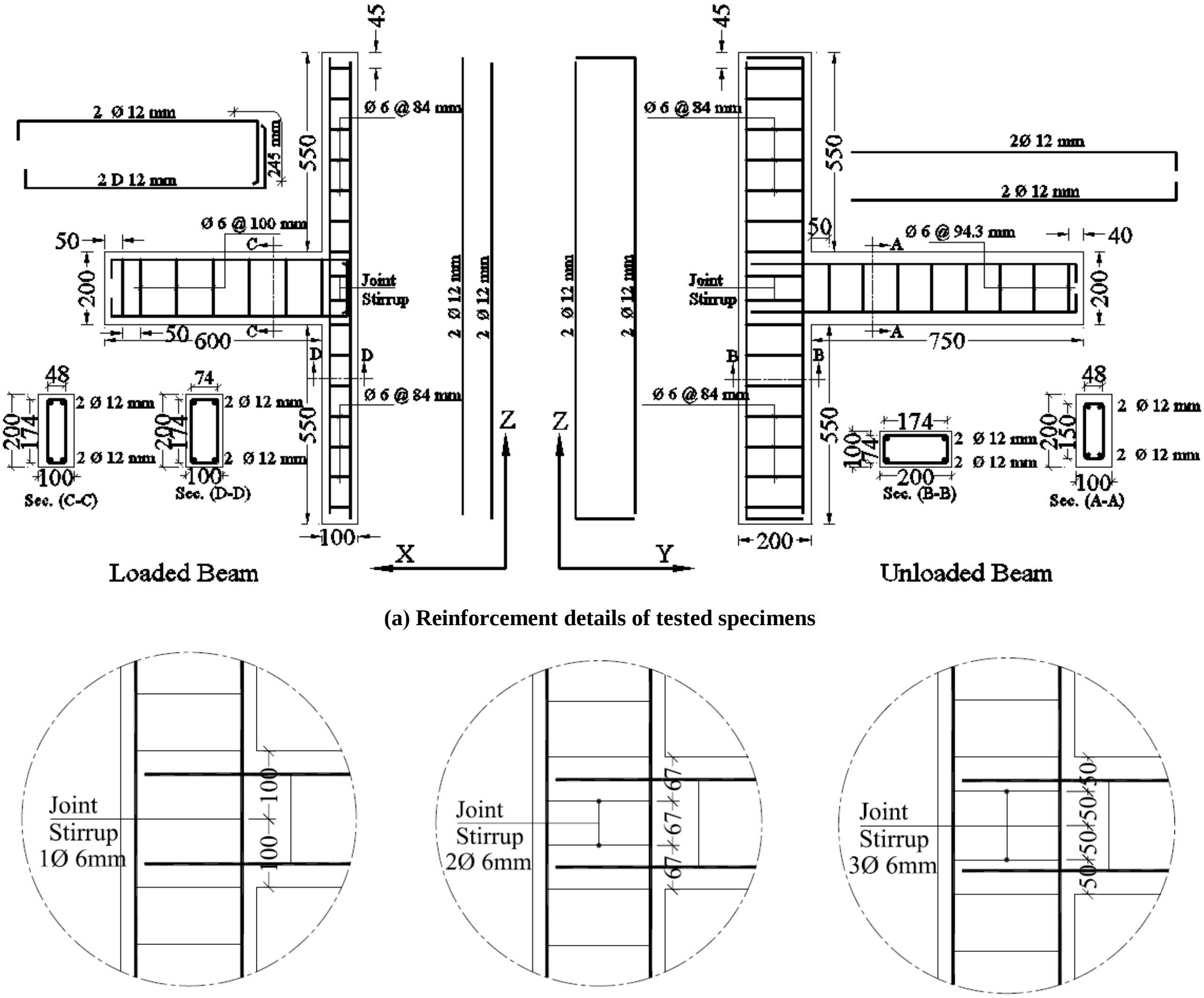
Material	Cement	Fine aggregates	Coarse aggregates	Water	Admixture (super plasticizer)
Weight (kg)/m^3	350	637	1295	175	3.5 liters

2.2. Material Properties

Concrete is a construction material composed of Portland cement and water combined with sand and crushed stone. Table 2 shows the components of the designed concrete mix by weight for one cubic meter. The concrete mix was designed to give an average cube crushing strength after 28 days equals to 35 MPa. For each specimen, three standard concrete cubes of size 150 × 150 × 150 mm were cast in order to estimate the accurate concrete compressive strength. The average compressive strength after 28 days was found equal to 35 MPa.

The unit weight of used sand was 1700 kg/m^3.Quarried, Crushed, and graded, dolomite is used as a coarse aggregate in the proposed concrete mix. The maximum size of crushed dolomite was 15 mm and its unit weight was 1600 kg/m^3.In order to attain an acceptable level of workability of the fresh concrete, superplasticizer (Sikament163M) was used.

Two types of steel reinforcement were used. High tensile steel (H.T.S) with a diameter of 12 mm and nominal yield stress of 410 MPa was used as main longitudinal reinforcement for the column and the beams. Normal mild steel with a diameter of 6mm and nominal yield stress of 250 MPa was used as transversal reinforcement for all parts of the specimen.



(a) Reinforcement details of tested specimens

(b) Horizontal stirrups for specimen P_1 **(c) Horizontal stirrups for Specimen P_2** **(d) Horizontal stirrups for specimen P_3**

Figure 3. Concrete dimension and reinforcement details of tested specimens

2.3. Test Set-Up Description and Instrumentation

The test set-up, shown in Figure 4, was used for the experimental investigation in the present research. The upper support of the column was hinged support in order to allow rotation about y-axes only. Bolt (1) and bolt (2) were added to the upper support and the lower support respectively in order to prevent the horizontal movement of the column in plan and out of plane as shown in Figure 4. The lower support was a roller to prevent the horizontal movement and it allowed the vertical movement. Three hydraulic jacks were used in this setup. Their capacity was 250 kN. The first hydraulic jack was used to apply 100 kN as a constant axial compression load on the lower end of the column during the test to represent the gravity load. The compression load was applied before the test and it was kept constant during the test. The second and the third jacks were used to apply the reversible cyclic load at the free end of the loaded beam A. The rate of loading was 50 N/minute. The original height of the column was 1300 mm and thickness of the upper and the lower support were 80 mm. So the effective height consider in the deformed shape is 1380 mm as shown in Figure 5.

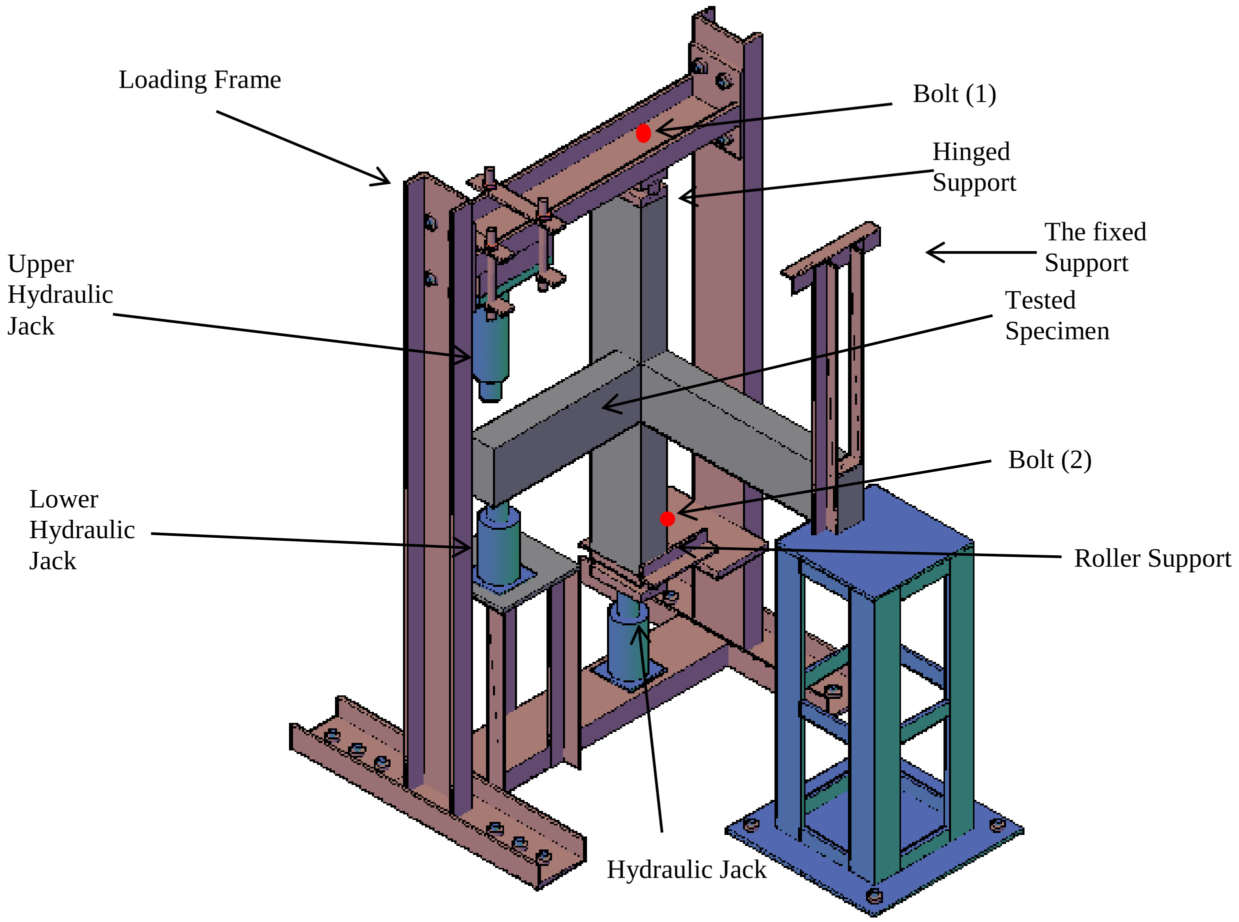
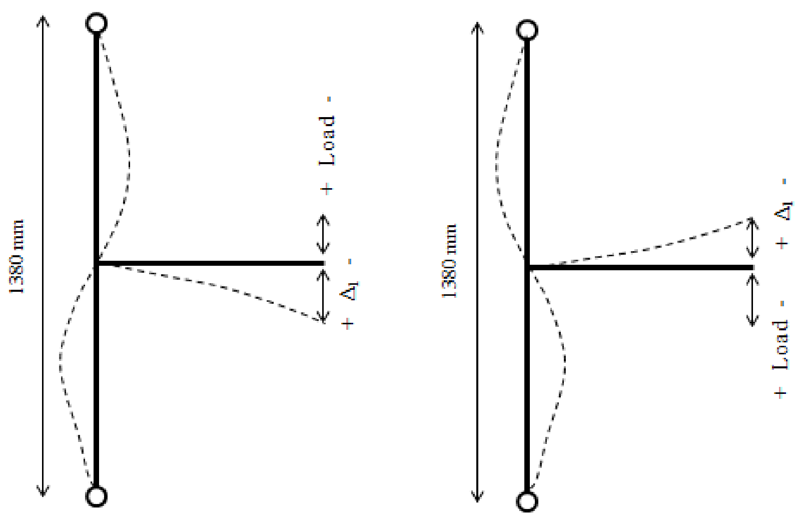


Figure 4. Three dimensional schematic diagram for the test set-up



(a) **(b)**

Figure 5. Details of instrumentation positions and predicted deformed shape; (a) down loading, (b) up loading

A dial gauge with a maximum displacement of 50 mm was used to measure the vertical deflection at the free end of the loaded beam A (Δ_1) as shown in Figure 5. Its accuracy was 0.01 mm. During the test, the vertical beam tip deflection and crack propagation were recorded at each cycle manually.

2.4. Cyclic Loading Sequence

The two hydraulic jacks were placed at end of the loaded beam A to apply the reversible cyclic loading. In the current study, the loading cyclic history is presented in Figure 6. The total numbers of cycles were14 cycle. Each cycle was larger than the previous one by 2 mm. The first cycle was divided into two stages. For the first stage, the free end of the beam loaded downward by the upper jack until the vertical displacement reached to 2 mm. The load of the upper jack was released. The lower jack was used to retain the beam to the initial position. For the second stage, the free end of the beam loaded up ward using the lower jack until the vertical displacement reached to -2mm. The load of the lower jack was released. The process was repeated for the following cycle but with increasing the displacement limit with 2 mm larger than the previous one.

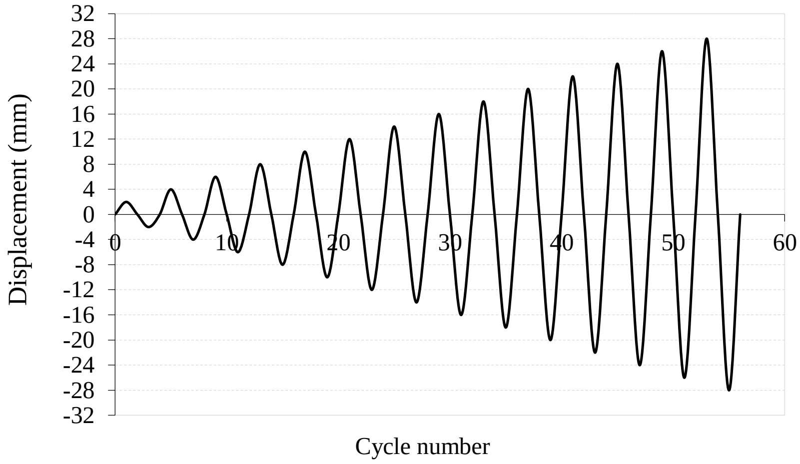


Figure 6. Cyclic loading histories

3. Experimental Results and Discussion

3.1. Mode of Failure and Crack Pattern

For reference specimen P_1, the cracks propagation was showed in Figure 7 in details. Inclined shear cracks and some flexure cracks occurred in the loaded beam (A) as shown in Figure 7a. These cracks were intersected together due to reversible loading. Some torsion cracks took place in the transverse beam (B) close to column edge as shown in Figure7b. Figure 7c showed the cracks propagation in the joint from the back side. The column was subjected to flexural stresses and this caused the horizontal cracks in the column as shown in Figure 7d. The failure was occurred at the joint panel due to small column depth in addition to the loaded beam (A) was not shifted from column edge. The mode of failure of $P_1$was shown in Figure 7a.Wide diagonal crack occurred at the front face of the joint at the joint with the loaded beam (A).

For specimen P_2, the cracks propagation and its distribution are shown in Figure 8. The first crack was classified as a flexural crack and it appeared in the loaded beam (A) close to the column face. The first crack occurred at the end of the first cycle. With increasing of the load, flexure cracks spread along the loaded beam as well as the joint. It was shown that diagonal cracks spread and it widened in the joint panel by increasing the reversible load. Small torsion cracks appeared in the transverse beam close to the column face. The main cracks started from beam to column edge. These cracks were intersected together due to reversible loading. The column was subjected to horizontal cracks in the column above and below the joint as shown in Figure 8e. The failure was occurred mainly due to horizontal cracks in the column above the joint. This specimen was tested up to 12 cycles only. The concrete cover of the column above the joint was crushed at the 11th cycle. At the 12th cycle, the specimen failed and the failure could be described as column hinging corresponded with more cracks in the joint.

For specimen P_3, Figure 9 shows details of cracks tracks for all over the specimen. The first crack was a flexure crack, appeared in the loaded beam, close to the column face, at the second cycle of loading. Flexure and shear cracks propagated in the loaded beam (A) as shown in Figure 9a. For the joint, diagonal cracks spread and widened by increasing the reversible load. The cracks of transverse beam were concentrated close to the column as shown in Figure 9b and 9c. For the column, horizontal cracks were propagated and spread above and below the joint as shown in Figure 9b and 9c. The crack pattern of the column side was shown in Figure 9d. The failure occurred due to the horizontal cracks widened in the column above the joint at the end of load cycle number 14, as shown in Figure 9e. The reasons of failure were illustrated as followed; (1) the column hinging above the joint, (2) vertical cracks inside of the joint and (3) inclined cracks in the joint from the back. It could be noticed that the increase in amount of the shear reinforcement transmitted the main failure into the column far away from the joint. Also, the cracks number increased and spread in a wide area.

(a) (b)

(c) (d)

Figure 7. Crack pattern of reference specimen P1; (a) beam (A) and joint (XZ plane), (b) beam (B), (c) the joint from back (YZ plane), (d) the column

3.2. Cracking and Ultimate loads

The effect of stirrups amount as shear reinforcement on the behavior of the beam column joint was studied. Values of the crack load, ultimate loads and ultimate deflections for all specimens were listed in Table 3. In this table, the values of the first cracks and ultimate load were labelled-- by P_{crl}, P_{crun}, P_{crf}, P_{crb}, P_{crs}, P_u and Δ_u. Where P_{crl} is the crack load of loaded beam due to flexure stress; P_{crun} is the crack load of transverse beam due to shear stress; P_{crf} the crack load of the front face of the beam-column joint due to shear stress; P_{crs} is the crack load of the side face of the beam-column joint due to shear stress; P_{crb} is the crack load of the back face of the beam-column joint due to shear stress; P_u is a peak value of the load deflection curve; and Δ_u is the deflection corresponding to P_u.

(a) (b) (c)

(d) (e)

Figure 8. Crack pattern of specimen P_2; (a) beam (A) and joint (XZ plane), (b) beam (B) and the column, (c) back of joint (d) side of the column. (e) specimen-bottom view

Table 3. First crack and ultimate loads of the tested specimens

Specimen	P_{crl} (kN) Loaded beam	P_{crun} (kN) unloaded beam	P_{cr} (kN) Beam-Column Joint			P_u (kN) Maximum load	Δ_u(mm) at Maximum load
			P_{crf} Front face	P_{crs} side face	P_{crb} Back face		
P_1	9	20	12	17	17	22.35	20
P_2	10	20	17	17	15	24	20
P_3	12	20	17	19	12	24	26

For all specimens, the first crack, P_{crl}, appeared in top surface of the loaded beam due to flexure stress. For P_2, the 1st crack appeared at P_{cr} of 10 kN. The first crack load, P_{cr}, of the loaded beam increased with the increase of the joint stirrups. The first torsion crack (P_{crun}) was initiated in the transverse beam at a load of 20 kN in all specimens. For all specimens, the first crack load, P_{cr}, of the joint occurred due to shear stress from the loaded beam A and transverse beam B. For both P_2 and P_3, the first crack load, P_{cr}, of the joint increased with the increase of the joint stirrups.

(a) (b) (c)

(d) (e)

Figure 9. Crack pattern of specimen P₃ ; (a) the joint and beam A, (b) the joint from the back, (c) beam B, (d) the column, (e) column hinging above join

The load carrying capacity of the reference specimen P_1reached 22.35 kN while it reached 24kN for both specimens $P_2$and P_3. When the joint shear reinforcement increased, the load carrying capacity of the joint increased. It was found that increasing the joint stirrups further has no significant effect on the load carrying capacity as the failure occurred in the column above the joint. The first crack of the joint from the back (P_{crb}) was affected by increasing of joint shear reinforcement. P_{crb} of specimen P_2 appeared at value less than of reference specimen P_1. It was may be because of the shear stress transferred to the back joint faster when the amount of stirrups increased. Also, it was noticed that the ultimate displacement (Δ_u) of the free end of the loaded beam at maximum load was affected by increasing of the joint stirrups. The Δ_u of P_3 was higher than Δ_u of $P_1$by about 30%. As the ratio of joint shear reinforcement increased from 0.36 % to 1.09%, the crack load of the joint from side (P_{crs}) increased also from 17 kN to 19 kN respectively. The result indicate that increase the horizontal stirrups of the BCJ there are noticeable improve in the seismic behavior and noticeable increase the joint shear strength.

3.3. Load Deflection Behavior

Load- Deflection curves were drawn for all cycles in Figures 10, 11 and 12 for P_1, P_2 and P_3 respectively. Deflection behavior of each specimen was studied then a comparison between the specimens was induced. The results showed that the slope of the curve in any current cycle was less than the curve slope in the previous cycle. As cycle number increased, the rate of deflection increased at the same level of loading. This may be due to decreasing of the rigidity with more cycles. Also, the cracks propagation in any cycle more than the previous was another reason. In the fact, the specimen had a low rigidity after the load release, so that all curves of specimens for all cycles resulted that the displacement value during load removing was bigger than the displacement value during load increase at the same load level.

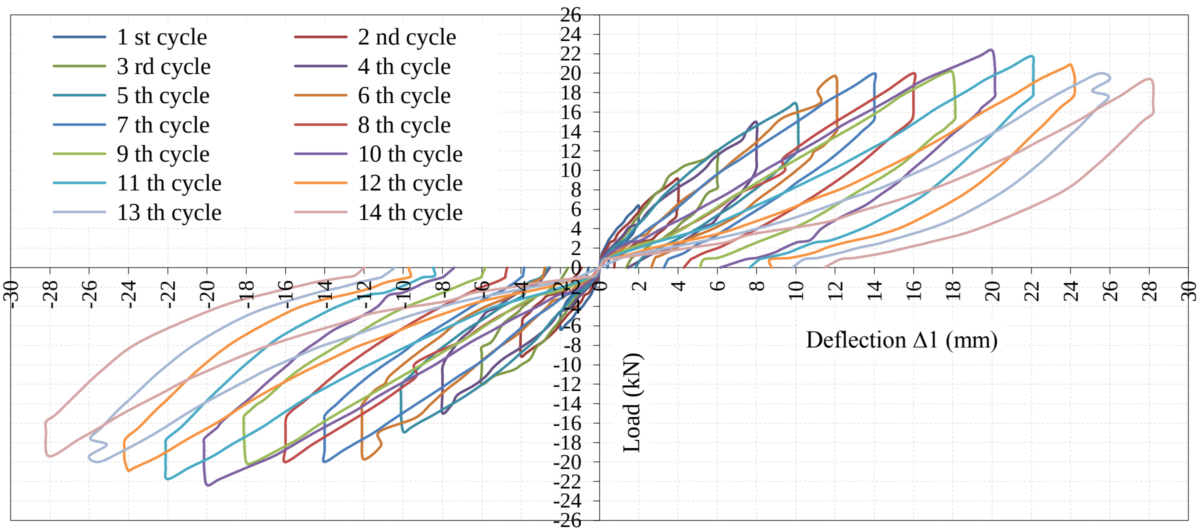


Figure 10. Load-deflection curve of P_1-Δ_1 for all cycles

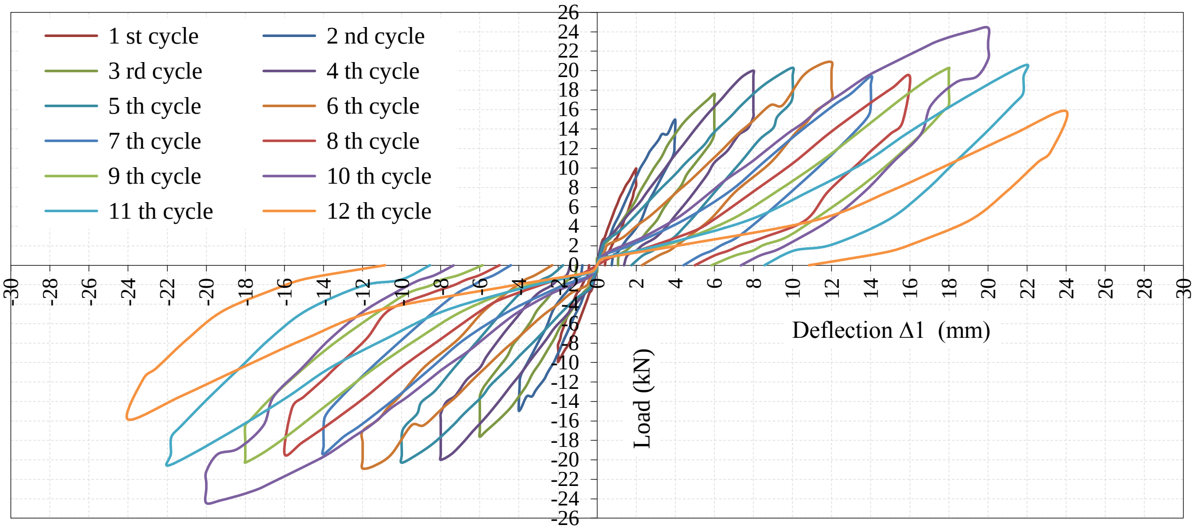


Figure 11. Load-deflection curve of P_2-$\Delta_1$for all cycles

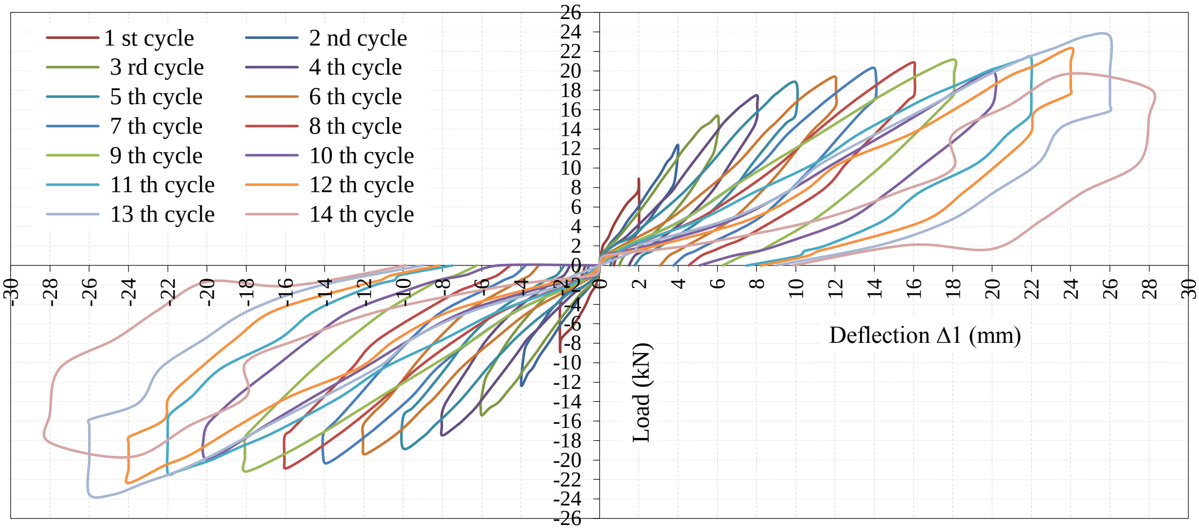


Figure 12. Load-deflection curve of P_3-Δ_1 for all cycles

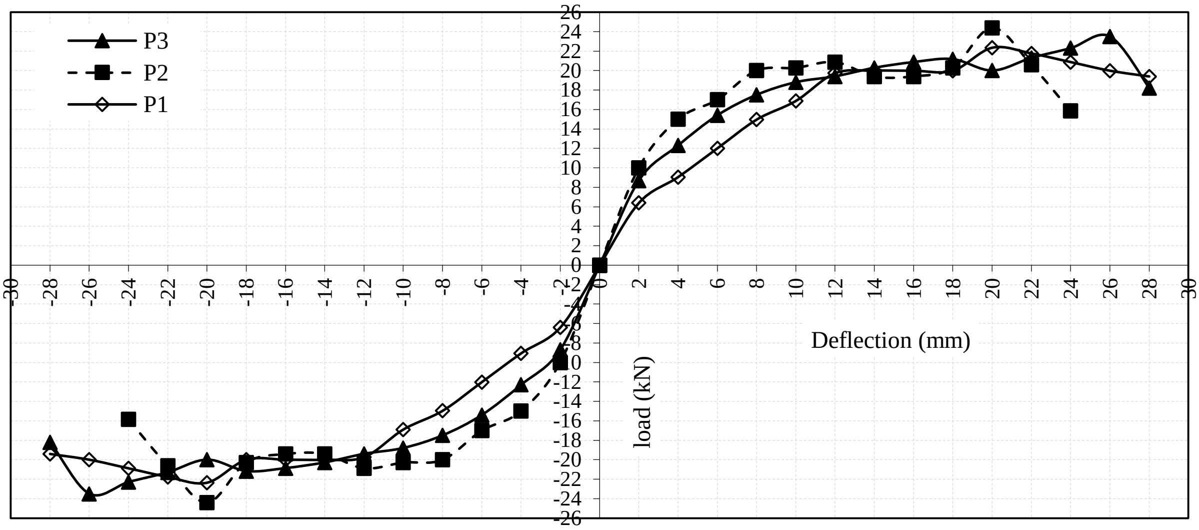


Figure 13. Load-deflection curves of the tested specimens -Δ_1 for all cycle peaks

For reference specimen P_1, fourteen cycles were carried out during the experiment. The load–deflection curves at each cycle were recorded. To make a self-comparison of one specimen, the load–deflection curve of $P_1$for all the 14 cycles was drawn in Figure 10. Each cycle was greater than the previous one by 2 mm.

All curves of P_1 for all cycles were showed in Figure 10where the deflection value during load release was bigger than the deflection value during load increase at the same load level. This was due to low rigidity of the specimen after loading. As a general trend; the percentage of the residual displacement decrease as the ratio of the joint shear reinforcement increase.

For specimen P_2, twelve cycles were carried out during the experiment. Figure 11 showed the load–deflection curves at each cycle for P_2. By comparing the two consecutive cycles, the slope of curve decreased with the increase in the loading. Also, the load at the cycle end increased until the 10th cycle then it reduced. From the 1st to 6th cycle, the curve had a linear relationship approximately.

For specimen P_3, the joint had 3Ø6 mm; the experiment was investigated under 14 cycles. The load–deflection curves at each cycle were recorded. The load–deflection curve of $P_3$for all cycles was drawn in Figure 12 in order to make a self-comparison of one specimen. P_3had the same behavior in the deflection that showed in P_2. It was showed that the slope of curve reduced with the loading. Also, the cycle's peaks increased until the 13th cycle.

The envelope of the load-deflection curve of the tested specimens (Δ_1) for all cycle peaks was drawn in Figure 13. The curve consisted of number of points, each point presented the peak of the cycle (displacement mm, load kN). All specimens had same behavior. The slope of the three curves was equal during the first step of the loading. From displacement 2 to 12 mm, both two specimens P_2 and P_3 was carried by the load more than the control specimen P_1. This may be because of the increase in the joint reinforcement. After the 12 mm displacement, the gradation of three specimens was a disturbance.

It was noticed that the residual displacement in any cycle was higher than the residual displacement in the previous cycle. Table 4 shows the percentage of the residual deflection to the maximum displacement at cycle end for all tested specimens. This might be due to cracks spread with the following cycles.

The yield displacement definition of experimental work is considered according to Ghani and Hamid [14] as shown in Figure 14. It was considered as the most practical definition for the yield displacement for the reinforced concrete structures. The displacement ductility is defined as the ratio between the ultimate displacement (Δ_u) to the displacement at first yield (Δ_y).From the test results, it was noticed that the ultimate displacement (Δ_u) is 28 mm. The average yield displacement for the three specimens was 8 mm. The displacement ductility is 3.50. The displacement ductility values are higher than 2.0 and this indicates that the design of tested specimen is adequate for earthquake loading.

Table 4. Percentage of the residual deflection to the maximum displacement at cycle end for all tested specimens ($\Delta_{residual}$ / Δ_{cycle} *100)

Cycle number	1st	2nd	3rd	4th	5th	6th	7th	8th	9th	10th	11th	12th	13th	14th
P_1	20.00	18.75	22.83	18.25	18.20	22.00	23.43	26.88	28.44	30.75	34.77	36.54	37.81	40.89
P_2	20.25	18.75	17.83	17.25	17.50	19.17	31.57	31.25	32.56	36.75	38.91	45.21	--	--
P_3	36.50	13.75	16.50	18.75	18.00	25.83	26.93	28.44	34.78	25.50	34.09	34.17	34.62	35.21

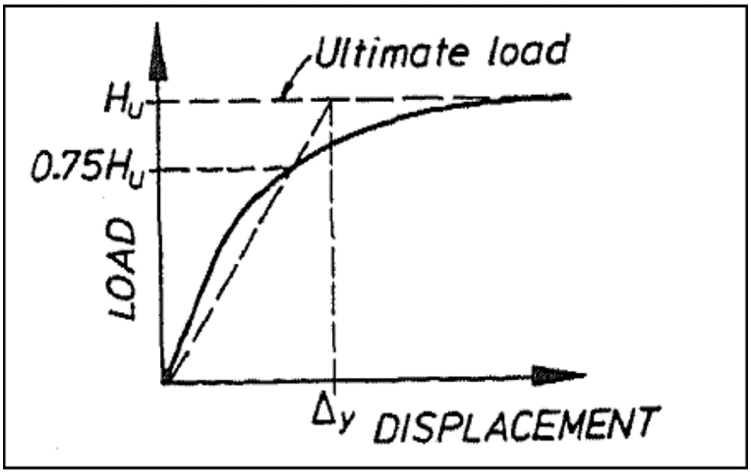


Figure 14. The realistic definition of yield displacement [14]

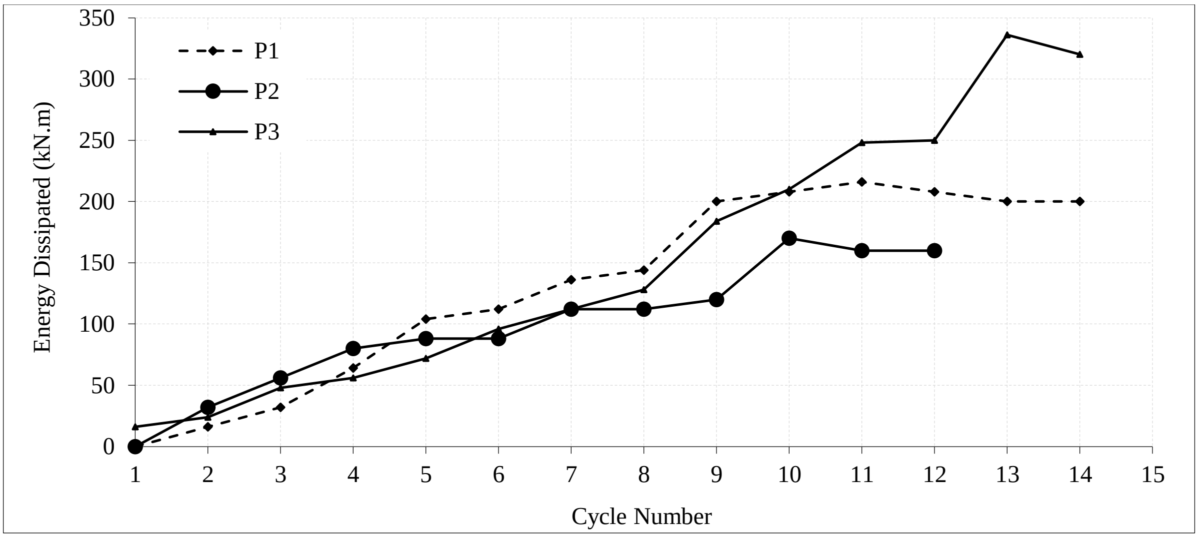


Figure 15. Energy dissipated of the tested specimens

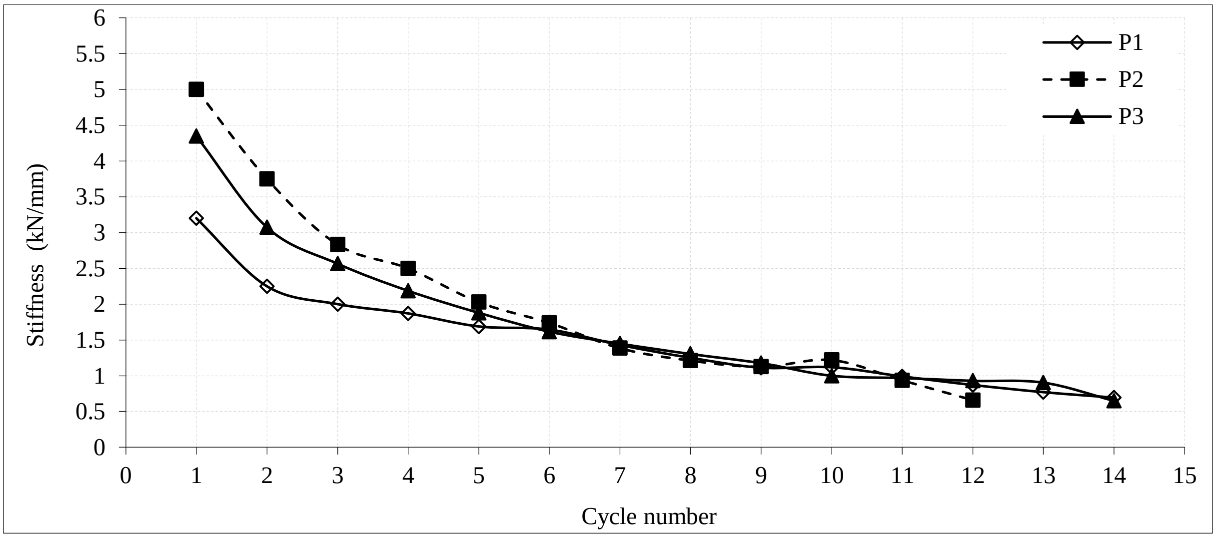


Figure 16. Stiffness degradation traces of the tested specimen

3.4. Energy Dissipation

The energy dissipated in each cycle was computed as the area enclosed by the load versus deflection-Δ_1 curves. The area inside each cycle was calculated in kN.mm. It was shown that the energy dissipated of the 1st cycle was zero approximately for all specimens. As the specimen was loaded by more cyclic load, the energy dissipated of cycle increased. For all specimens, Figure 15 showed the rate of the increase in the energy dissipated for all cycles. For specimenP_1, the increase in energy dissipated at the test start was larger than the increase in the energy dissipated during the test. At the 11th cycle, the energy dissipated began in decreasing. For P_2, the uniform increase was occurred in the energy dissipated from the start to the end except for the 10th cycle. The energy dissipated was the maximum value at the 10th cycle at the ultimate load of the specimen P_2.For P_3, the energy dissipated had a uniform increase until the 8thcycle. The maximum value of the energy dissipated took place at 13th at the maximum capacity of this specimen P_3. The cumulative energy dissipation increased by 75% for the specimen P_3 than the specimen P_1. On the other hand; the cumulative energy dissipation decreased by 20% for the specimen P_3 than the specimen P_1. The stirrups distribution inside the beam column connection has a great effect on the energy dissipation and the crack pattern. Where the experimental results indicate that the stirrups are most effective in is the middle third part of the BCJ than others two parts.

3.5. Stiffness Degradation

The stiffness of the beam-column joint under cyclic load can be defined as it resists deformation due to an applied force. Figure 16 present the relationship between beam column stiffness with respect to cycle number for all specimens. In this study, the stiffness of the corner beam-column joint under cyclic load is obtained by divide peak load of the cycle to its corresponding deflection measured in the experiment. The result indicates that the relationship between beam-column joint stiffness and cycle number is an inverse relation. The residual stiffness depends on the damage degree. As the stirrups joint percentage increase, the stiffness of beam column joint increase. There are identical between three curves after the cycle number 6.

Based on the experimental result; the stirrups increased the joint shear capacity and restrained the deformations. As the stirrups percentage increase the crack load increase and the crack appear in a late stages in the back face of the beam-column joint as stated by Marthong et al. [12].

4. Conclusion

An experimental work was carried out to study the behavior of RC eccentric beam-column joint under cyclic loading. An experimental study that consisted of three specimens was investigated in the current research. The specimen consisted of two columns (upper and lower parts) and two beams one is the free end (loaded beam A) and the other is the fixed end (transverse beam B). The main objective is the amount of the reinforcement of the joint. The specimens were tested under reversible 14 cycles. The vertical deflection at free end of loaded beam and cracks propagation were recorded at each cycle. Also, the crack patterns, crack propagation, failure modes were studied. Based on the investigation carried out in this research, the following conclusions may be drawn.

- It was found that the 1st crack on the front joint side appeared notably earlier than the back joint side.
- It was noticed that the increase in the amount of the stirrups of the joint transmitted the main failure into the column faraway the joint. Also, the cracks number increased and it spread in a large area of the joint.
- For both P_2 and P_3, those were reinforced by more stirrups in the joint, the 1st crack of the joint from front delayed with respect to the reference specimen P_1 due to the increase of the stirrups.
- When the joint reinforcement was increased, the capacity of the joint increased.
- At same displacement, specimens P_2 and P_3 that were reinforced by more stirrups in the joint were carried by the load more than the control specimen P_1.

5. Conflict of interest

The authors declare no conflict of interest.

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