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The effects of stirrups and the extents of regions used SFRC in exterior beam-column joints

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Abstract. Seven full-scale exterior beam-column joints were produced and tested under reversible cyclic loads to determine. Two of these seven specimens were produced using ordinary reinforced concrete (RC). Steel Fiber Reinforced Concrete (SFRC) was placed in three different regions of the beams of the rest five specimens to determine the extent of the region where SFRC is the most effective. The extent of the region of SFRC was kept constant at the columns of all five specimens. Three of these five specimens which had one stirrup in the joint, were tested to evaluate the effect of the stirrup on the behavior of the beam-column joint together with SFRC. In production of the specimens with SFRC, all special requirements of the Turkish Earthquake Code related to the spacing of hoops were disregarded. Previous researches reported in the literature indicate that the fiber type, the volume content, and the aspect ratio of steel fibers affect the behavior of beam-column joints produced with SFRC. The results of the present investigation show that the behavior of exterior beam-column joints depends on the extent of the region where SFRC is used and the usage of stirrup in the joint, in addition to the parameters listed in the literature.

Keywords: beam-column joint; reversible cyclic loading; steel fiber; concrete, energy capacity; ductility; earthquake.

1. Introduction

During strong earthquakes, beam-column joints are exposed to higher moments and shear forces than the other regions of beams and columns. Sudden degradation of strength and stiffness of reinforced concrete (RC) lateral load resisting moment frames due to joint damages should be eliminated because the joint is a part of the column and its damage directly affects the overall response and stability of such frames. The ductile behavior of RC frames primarily depends on the material characteristics, the reinforcement detailing of beam-column joints and beam plastic hinge region, and the amount of beam and column hoops. Due to the significant contribution of joint failures to the collapse of buildings during earthquakes, it is necessary to prevent a brittle shear failure and to upgrade the joint's load carrying capacity and ductility. To ensure these actions should be developed both of economical materials and laborsaving reinforcement details for the construction of beam-column joints. Previous investigations on beam-column joints by Pessiki

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(1990), Kurose *et al*. (1988), Kitayama *et al.* (1991), Aoyama (1985), Fuji and Morita (1991), Paulay *et al.* (1989) and Paulay (1989) show that the shear strength and ductility of RC beam-column joints increase as the compressive strength of the concrete and the amount of transverse reinforcement increase. Tsonos (2004) reported that axial load and P-Delta effect during earthquake cause significant deterioration in the earthquake-resistance of these structural elements and inclined bars in the joint region were effective for reducing the unfavorable impact of the P-Delta effect and axial load changes in the structural elements. Hakuto *et al*. (2000) indicate that the seismic performance of RC frames without transverse reinforcement in the beam-column joint core would be poor in a severe earthquake and the available shear strength of interior beam-column joint cores without transverse reinforcement decreases with increase in the imposed ductility of the adjacent beam plastic hinge regions. Test results performed by Hwang *et al*. (2005) indicate that the major function of the joint hoop is to carry shear as a tension tie and to constrain the crack width. The results of the tests performed by Chutarat and Aboutaha (2003) indicate that headed bars can be used to relocate the plastic hinge region and give good results in higher shear forces in the beam-column joints. "The Recommendations of the Specification for Structures to be built in Disaster Areas in Turkey (Turkish Earthquake Code, TEC)" states requirements almost similar to the ACI 318-02 from the viewpoint of column and beam flexural strength, the requirements of shear strength and hoop spacing, reinforcement detailing and limiting amount of reinforcement. Both codes propose similar requirements so that RC frames have high ductility and are able to dissipate seismic energy without collapse during severe earthquakes. The requirements of the TEC for the design and the spacing of beam, column and joint hoops are the same with those of ACI 318-02. At the beam-column joints subjected to high bending moments and shear forces, the plastic hinges must form in the beam sections rather than column sections. Consequently, the sum of bending moment capacity of columns has to be higher than the sum of bending moment capacity of beams at the beam-column joints. Beam-column joints usually have congestion of reinforcements due to the hoops and excessive amount of longitudinal reinforcement due to high bending moments in beams and columns. Therefore, workers tend to avoid placing the hoops in beams, columns, and joints. Especially in inspections on damaged RC structures in the Kocaeli and Bolu-Duzce earthquakes in 1999 in Turkey revealed that beam and column hoops were not placed in beam-column joints in accordance with TEC. Numerous studies have been carried out to decrease the reinforcement congestion in the joints and to develop laborsaving reinforcement details and materials. One of these attempts is to use SFRC instead of the hoops in columns, beams, and joints. However, addition of steel fibers of higher volume fractions into concrete mix may reduce the workability and may cause the balling of steel fibers in the mixture. This problem can be solved by adding some superplasticizer into the concrete mix and by limiting the maximum size of coarse aggregate up to 10 mm.

In the previous studies on the use of SFRC performed by Jindal and Hasan (1984), Craig *et al.* (1984), Jindal and Sharma (1987), Sood and Gupta (1987), Katzensteiner *et al*. (1992), Jiuru *et al.* (1992) performed test by using SFRC in any region of beam-column joints. The results of reverse cyclic loading tests on full scaled exterior beam-column joint specimens performed by Filiatrault *et al*. (1994 and 1995) show that steel fibers bridging across cracks in the concrete mix increase the joint shear strength and reduce or even eliminate necessity for closely spaced ties. The common results of these researches show that the use of SFRC instead of hoops in the beam-column joints does not only decrease the cost of material and labor but also increases the ductility and strength of the joints. In the first stage of the present experimental investigation, four tests performed by

Gencoglu and Eren (2002) indicate that SFRC could be used instead of hoops, and an increase in the load carrying capacity, the amount of the total absorbed energy and the ductility of the exterior beam-column joint specimens could be accomplished. Bayasi and Gebman (2002) also reported that reduction of lateral reinforcement in seismic detailed beam-column joints was possible with the appropriate use of SFRC and such a reduction can help in laborsaving reinforcement details and decreasing steel congestion at the beam-column joint. Singh and Kaushik (2002) show that the addition of steel fibers in the matrix modifies the bond characteristics of the reinforcement bars, and a higher diagonal steel content, equal to the main tension reinforcement, could be effectively used in the RC corner beam-column joints. Shannag *et al.* (2005) reported that using high performance steel fiber reinforced concrete in place of ordinary concrete in the joint region of non-seismically designed beam-column joints improved significantly its seismic behavior.

2. Research significance

It is worth to mention that the behavior of the beam-column joint comes into being as a sum of the behavior of the beam and the column and the joint itself. Consequently, these parts of structures can be also named as "beam-column-joint assemblage". The overall behavior of this assemblage is of prime importance to predict the failure mechanism of a frame. Generally, the joint is the weak part of this chain. It is why the present paper focuses on the behavior of the joint. Previous experimental investigations studied the effects of SFRC on the behavior of beam-column joints by varying hoops spacing, type and aspect ratio, amount of fibers, action points of the cyclic loads, and scales of specimens as the experimental parameters. In this research, SFRC was placed within three different regions of beam of exterior beam-column joints to evaluate the effect of the length of the region with SFRC on the behavior of beam-column joints. Additionally, only one stirrup was placed into the joint of three specimens to investigate the effects of the transverse reinforcement when stirrup was used together with SFRC in the joint. The experimental results of this research were evaluated from in terms of the amount of total, elastic recovery, and dissipated energy capacities, damage distribution of specimens, and flexural rigidity variations of plastic hinge zones of the beams in the beam-column joints. Based on these results, the effect of length of the region with SFRC was evaluated. Seven full-scale exterior beam-column joint specimens for this research were tested under the reversible cyclic loads to simulate earthquake forces.

3. Description of test specimens

A prototype four-story school building was completely designed for the seismic zone with the highest risk according to TEC. The specimens tested in the present experimental investigation represent the first floor exterior beam-column joint of this four-story school building. The first specimen designated as RCTEC was designed and produced according to the TEC requirements for the spacing of hoops in beam, column and joint by using ordinary RC. The specimen designated as RCNHJ was also produced using ordinary RC according to the requirements of the TEC but no transverse reinforcement was provided. For other five specimens, SFRC was placed in the joint, the beam regions ($2h_{beam}$, h_{beam}, and $1/2h_{beam}$ from the column face), and the confinement region of column. The rest regions of the beam and column of these five specimens were made of ordinary

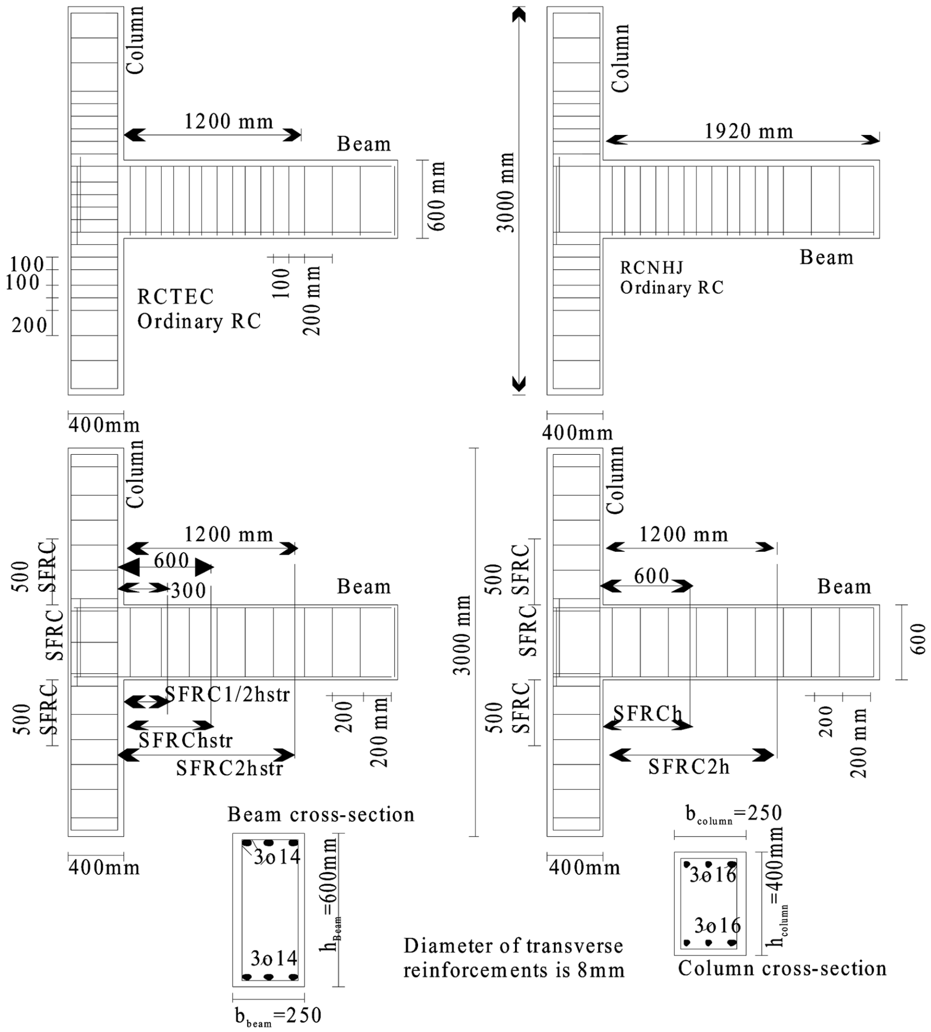

Fig. 1 Geometry and details of the test specimens

RC. Only one stirrup was placed into the joints of three specimens. These three specimens were designated as SFRC2hstr, SFRChstr, and SFRC1/2hstr based on the extent of the region that has SFRC and the usage of stirrup in the joint. The remaining two specimens with SFRC did not have any stirrups in the joint. These specimens were designated as SFRC2h and SFRCh. In these specimens, SFRC was used instead of beam, column and joint hoops. The geometry and details of the test specimens are given in Fig. 1.

The steel mold was placed on the laboratory floor horizontally and the concrete was cast into the mold by avoiding the mixing of plain concrete and SFRC with each other. Both the beam and the column were cast in a single operation. Vibration was applied after casting the concrete to compact the concrete adequately.

Table 1 Characteristic properties for the concrete mixtures

Materials	Unit	Plain concrete	SFRC
Cement	kg/m^3	450	450
Aggregate-1 (10 mm)	kg/m^3	1210	1210
Sand	kg/m^3	465	465
Water	kg/m^3	265	265
Superplasticizer	ml/m^3	5000	5000
Steel fiber 60/0.8	kg/m^3		78

4. Material properties and concrete mixes

Two different ready-mixed concrete designs were used. A summary of the mix designs is given in Table 1. The compression test results of the concrete cylinders revealed that the average compressive strength of the plain concrete and the SFRC were 29 MPa and 24 MPa, respectively. According to the uni-tension tests performed in the laboratory, the yield stress of the transverse and longitudinal ribbed reinforcement was 500 MPa. The collated hooked-end steel fibers having a length of 60 mm, a diameter of 0.8 mm (aspect ratio of 75) and a yield stress of 1100 MPa were added into the plain concrete mix at 1% by volume except for SFRCh. Steel fiber content of SFRCh was 2% by volume. The maximum size of coarse aggregate was limited to 10 mm. Superplasticizer was added into the concrete mixture. So that SFRC would easily be placed into the mold.

5. Experimental set up and testing procedure

The tests were carried out in the Structure & Earthquake Laboratory of Civil Engineering Faculty of Istanbul Technical University, Turkey. In the experimental set up, the test specimens were placed to the loading frame such that the columns were horizontal and the beams were vertical position. In other words, the vertical element represents the beam, and beam tip end corresponds to mid-span where the bending moment is nearly zero during an earthquake. The reversible cyclic loads were

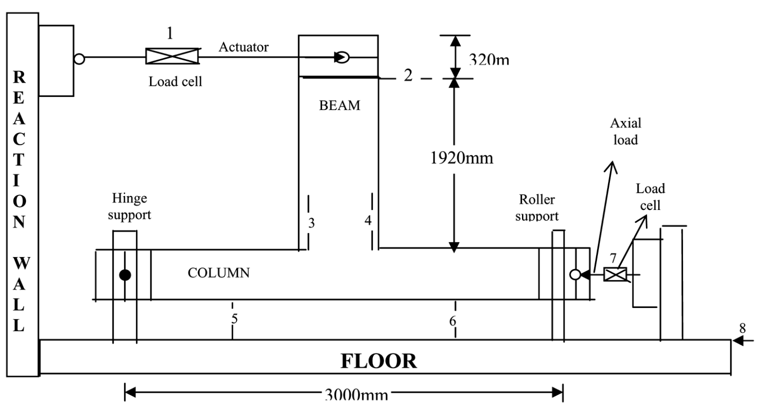

Fig. 2 General arrangement of the experimental set up and the beam tip displacement transducers

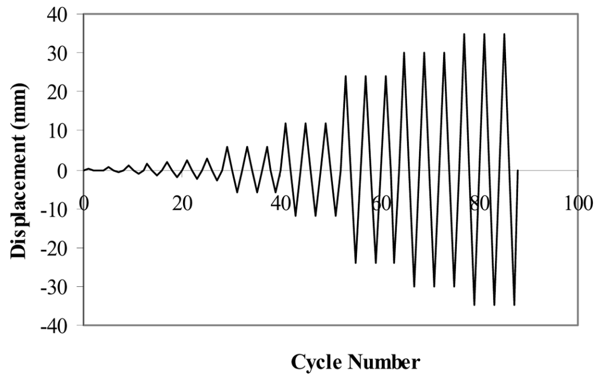

Fig. 3 Displacement controlled cyclic loading

applied to the end of the beam by controlling the tip displacement of the beam. The configuration of the test specimens is given in Fig. 2. The length of the column at the experimental set up was determined by taking the inflection points of column into consideration.

Tests performed by Bonacci and Pantazopolou (1993), Fuji and Morita (1991 revealed that axial force of the column does not affect overall behavior and strength of beam-column joints. However, Paulay (1989) and Paulay *et al.* (1989) expressed that axial force of column has causes an increase in the shear strength of beam-column joint by balancing the diagonal concrete compressive stress induced by column shear forces at joint face. According to the results of these researches, at the beginning of all tests, an axial load of 250 kN ($0.1A_g f_c'$) was applied and the magnitude of this axial load varied depending on the direction and magnitude of reversible cyclic loads applied to the beam tip. The cyclic loads were applied by using horizontal double acting hydraulic actuator, located on the beam at a distance of 2.24 m from the face of the column. The reversible cyclic loads were used to simulate an earthquake motion as shown in Fig. 3. The first three cycles were established in the elastic range and all the rests of the cycles were in the inelastic range and three cycles were performed for each peak displacement level in the inelastic range, until failure. Displacement transducers (LVDT) were placed at six different points on each specimen to measure the deformations and the displacements of the beam-column joint specimens as shown in Fig. 2. Load cells were used to measure loads applied from the actuators to the specimens. For each loading cycle, the signals taken from the displacement transducers and load cells were recorded by a personal computer using a data acquisition system until the target displacement level of each cycle was reached. Furthermore, the propagation of cracks was marked on the specimens and recorded by a camera.

6. Experimental results

6.1 General behavior and failure mechanism

The horizontal load-tip displacement hysteresis loops of the beam were obtained from the experiments for all of the specimens. Two of these loops are shown in Figs. 4(a) and (b) for RCTEC and SFRC2hstr, respectively. Additionally, the peak load - beam tip displacements envelopes of the beam-column joints were produced based on the peak values of each load-

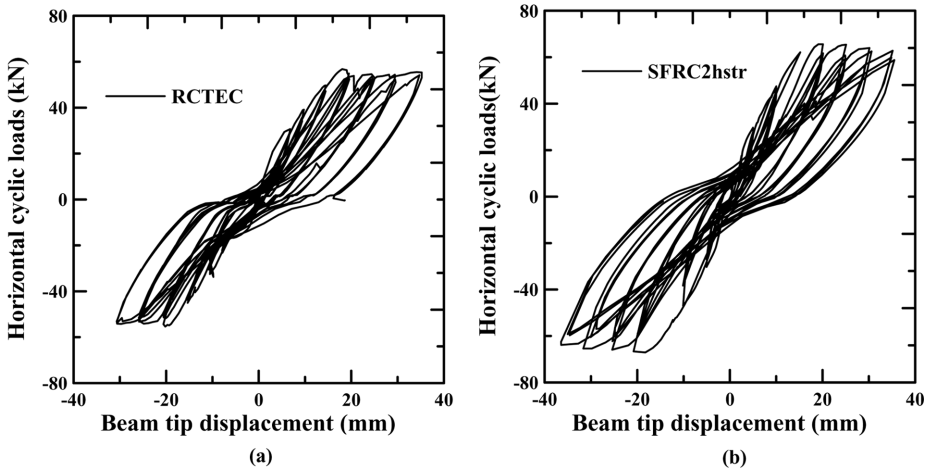

Fig. 4 Horizontal loads versus tip displacement hysteresis loops of the beam for RCTEC and SFRC2hstr

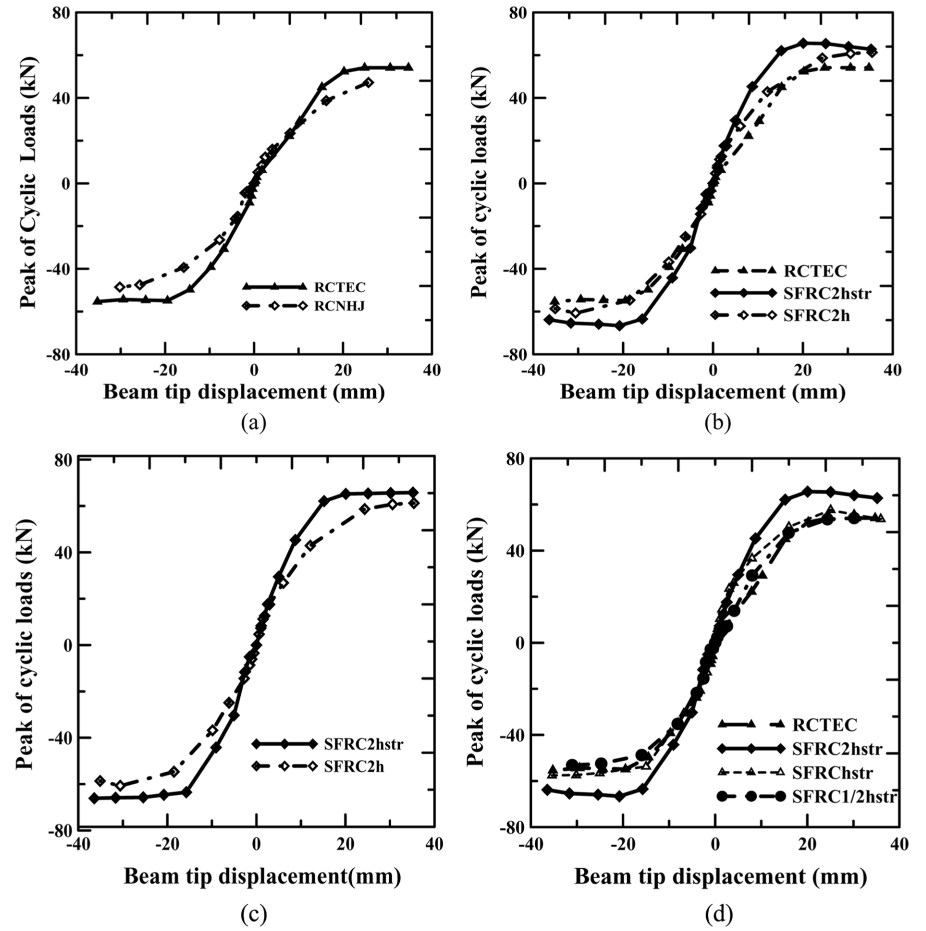

Fig. 5(a)-(d) Horizontal reversible cyclic loads versus tip displacements envelopes of the beam for selected specimens displacement hysteresis loop measured at the beam tip. These envelopes are presented in groups in Figs. 5(a)-(d) to illustrate the effects of the extent of the region with SFRC, the usage of stirrup with SFRC in the joint and joint hoops in RC specimens. Results from Figs. 5(a)-(d) can be stated as follows:

1. In the design of RC structural elements, the flexural failure mode is preferred rather than the brittle shear failure mode. All the specimens, except for SFRC1/2hstr failed by widening of the beam flexural cracks at the column face. Although SFRC1/2hstr had almost the same cyclic load carrying capacity as RCTEC until failure, SFRC1/2hstr failed earlier than RCTEC by excessive widening the flexural crack at the junction of SFRC and ordinary RC of beam.
2. The peak loads of all specimens were almost equal at the same displacement levels until the first structural crack occurred. Following the load level in which the first structural crack occurred, Figs. 5(b) and (d) indicate that SFRC and only one stirrup in the joint with SFRC increase the cyclic load carrying capacity of beam-column joints.
3. RCNHJ, which has no hoops in the joint, shows the most brittle behavior one among all the specimens (Fig. 5(a)). Consequently, the cyclic load and displacement level where RCNHJ failed was less than other specimens. Although at failure, RCNHJ had wider shear cracks than SFRC specimens in the joint, the excessive widening of beam bending crack at the column face of joint caused the failure. When the behavior of RCNHJ is compared to that of the RCTEC (Fig. 5(a)), it can be concluded that the joint hoops are of prime importance for beam-column joints made of ordinary RC. TEC and ACI 318-02 recommend to be placed the hoops into the joint to increase the strength and the ductility of beam-column joints.
4. All specimens, except RCNHJ and SFRC1/2hstr, carry the reversible cyclic loads until a tip displacement level of 35 mm. Both RCNHJ and SFRC1/2hstr reached flexural failure mode at the third cycle at a displacement of 30 mm.
5. Failure load of SFRC2hstr was 20% higher than that of the RCTEC. The maximum load carrying capacity of SFRC2hstr was also 10% higher than that of SFRC2h (Fig. 5(b)). These results show that SFRC can be used instead of hoops in the beam-column joints.
6. In Fig. 5(c), the results of SFRC2hstr and SFRC2h are given comparatively to show the effect of the usage of a single stirrup together with SFRC in the joint. The load carrying capacity of SFRC2hstr was 10% higher than that of the SFRC2h. It indicates that the strength and behavior of the beam-column joints with SFRC can be improved by placing one stirrup at least into the joint.
7. Effect of the extent of the region with SFRC on the behavior of beam-column joints are shown in Fig. 4(d). SFRC2hstr had the highest reversible cyclic load capacity among all the specimens. Fig. 4(d) further indicates that SFRC1/2hstr had a reversible cyclic load capacity comparable to that of RCTEC detailed according to the TEC requirements until the ultimate displacement level of 30 mm. However, SFRC1/2hstr failed earlier than the other SFRC specimens and RCTEC due to widening of the flexural crack at the junction of SFRC and ordinary RC. The reason for the behavior of SFRC1/2hstr might be due to the difference in flexural rigidity of the section of SFRC and ordinary RC.
8. Although the spacing of the stirrups in the specimens with SFRC was larger than the other two specimens completely made of ordinary RC, the specimens with SFRC had larger load carrying capacity and displacement levels than RCTEC and RCNHJ, except for SFRC1/2hstr. Based on these results, it can be concluded that SFRC increases the load carrying capacity of beam-column joints subjected to reversible cyclic loads.

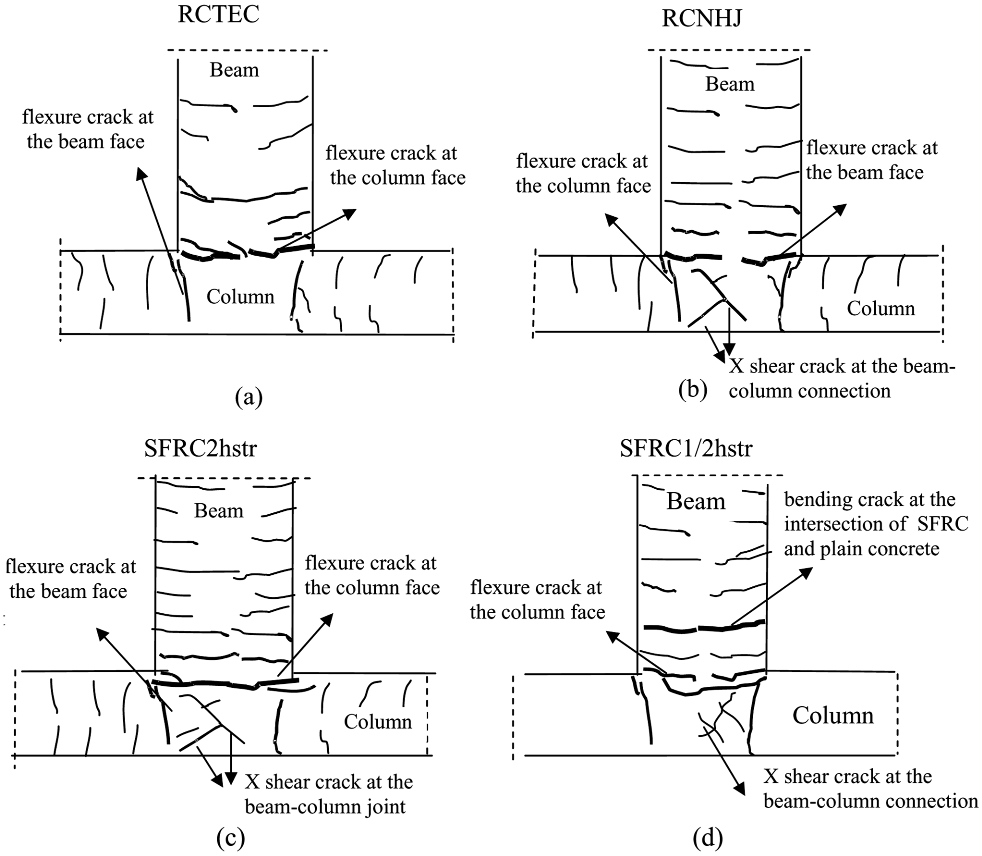

Fig. 6 Crack propagation of some specimens in the joint and the confinement regions of the beam and column

6.2 The shear cracks and stresses of the exterior beam-column joints

The crack propagation in all specimens tested under reversible cyclic loads was similar, except for RCTEC. The "×-type" shear cracks occurred at the joints of all the specimens which did not have hoops in the joint. The crack propagation of some specimens is shown in Fig. 6. On the other hand, the × shear cracks in RCNHJ occurred earlier than those in the SFRC specimens. While the maximum width of the "×-type" shear cracks in the joints of SFRC specimens was almost 0.3 mm, the cracks had a maximum width of 1~1.5 mm in RCNHJ. The use of SFRC did not hinder the occurrence of the two sided "×-type" shear cracks in beam-column joints, but the enlargement of the width of these cracks could be prevented by steel fibers. Steel fibers in the concrete mixtures lessen the width of the bending or shear cracks by bridging their two sides and transferring stresses between them. These cracked sections continue to carry the internal forces until the steel fiber slips from the concrete. The joint region is subjected to excessive shear stresses, when any of the adjoining members exceed its strength moment capacity associated with the hardened plastic hinge. For the exterior beam-column joints, the horizontal and vertical joint shear stresses (τ_{jh}, τ_{jv}) is given as (Murty *et al.* 2003)

$$\tau_{jh} = \frac{H}{A_{core}^{h}}\left(\frac{L_b}{d_b} - \frac{L_b + 0.5D_c}{L_c}\right) \tag{1a}$$

$$\tau_{jv} = \frac{H}{A_{core}^{v}}\left[1 - \left(\frac{L_b + 0.5D_c}{L_c}\right)\left(\frac{L_c - D_b}{d_c}\right)\right] \tag{1b}$$

where H is reversible cyclic loads at the beam tip; L_b and L_c are lengths of beam and column, D_b and D_c are total depths of beam and column, d_b and d_c are effective depths of beam and column, A_{core}^{h} and A_{core}^{v} horizontal and vertical cross-sectional area of the joint core resisting the horizontal and vertical shear forces, respectively.

The parameters that affect the shear strength of the beam-column joints are: 1- transverse reinforcements in the joint, 2- physical size (i.e., volume) of the joint, 3- members framing into the joint, and 4- adequate anchorage of longitudinal beam bars in the joints. When joint shear forces become large, diagonal cracking occurs in the joint core followed by crushing of concrete. It is necessary to limit the magnitude of horizontal joint shear stress to prevent such diagonal cracks in the joint. Therefore, codes such as ACI 318-02 and NZS 3103 require that shear stresses in the joint core are kept below a maximum permissible value.

$$\tau_{jh} = k\sqrt{f_c'} \quad \text{in accordance with ACI 318-02} \tag{2a}$$

$$\tau_{jh} = 0.20f_c' \quad \text{in accordance with NZS 3103} \tag{2b}$$

where f_c' is the cylinder compressive strength, MPa, k factor depends on the confinement provided by the members framing into the joint; k is taken as 1.67, 1.25, and 1.0 for interior, exterior and corner joints, respectively. However, TEC requires the following for shear safety of the exterior beam-column joints requires

$$V_e = 1.25f_yA_s - V_{column} \tag{3a}$$

$$V_e \leq 0.30b_jhf_c' \tag{3b}$$

where, f_y is the yield stress of beam longitudinal reinforcement; A_s is the total cross-sectional area of beam tensile longitudinal reinforcements; V_{column} is the smaller one of the shear forces above and below the joint due to earthquake loading; b_j is twice smaller of the distances measured from the vertical centerline of a beam framing into the beam-column joint in the earthquake direction, to the edges of column (shall not exceed the beam width plus the joint depth); h is column cross section dimension in the earthquake direction considered.

Eqs. (1a) and (1b) indicate that the horizontal and vertical joint shear stresses depend on the magnitude of the cyclic load and the physical size of the joint. In this research, the joint sizes of all specimens were kept constant. Fig. 5(b) shows that SFRC2hstr and SFRC2h carried greater reversed cyclic loads than RCTEC. Therefore, SFRC increases the shear strength of the beam-column joints subjected to reversible cyclic loads. Although "x type" shear cracks occurred in the joints of SFRC specimens, the width of these cracks were limited to 0.3 mm and the corresponding damages were not the prime reason which leads the failure of the specimens.

6.3 Energy capacity

The ductility of the specimens was also evaluated by taking into account the total energy absorbed by the beam-column joint assemblies under reversible cyclic loads. The test results in this research were evaluated to determine the ductile level of the beam-column joint of the test specimens from viewpoint of the total, dissipated, and recovery energy capacities of ones. The amount of the accumulated hysteretic energy of a joint was calculated as the area under the curve of the horizontal force-beam tip displacement hysteresis loop up to the corresponding displacement level as follows

$$W_i = \int_{x_1}^{x_2} P_{push}(x)dx + \int_{x_3}^{x_4} P_{pull}(x)dx \tag{4}$$

$$W_{total} = \sum_{i=1}^{n} W_i \tag{5}$$

$$W_i^{recv} = \int_{x_2}^{x_3} P_{push}(x)dx + \int_{x_4}^{x_5} P_{pull}(x)dx \tag{6}$$

$$W_{recovery} = \sum_{i=1}^{n} W_i^{recv} \tag{7}$$

$$W_{dissipated} = W_{total} - W_{recovery} \tag{8}$$

where,

n : the total cycle number
W_i : the accumulated energy at the ith cycle
W_i^{recv} : the elastic recovery energy at the ith cycle
W_{total} : the total energy absorbed by specimen
$W_{recovery}$: the total elastic recovery energy absorbed due to elastic behavior of specimen
$W_{dissipated}$: the total energy dissipated by plastic behavior of specimen
$P_{push}(x)$ and $P_{pull}(x)$: the push and pull cyclic loads depend on xi, displacement of the beam tip, respectively.

These variables and the algorithm used in the evaluation of the accumulated hysteretic energy are shown in Fig. 7 for any ith cycle of a beam tip load-displacement hysteresis loop model of a beam-column joint specimen. At the end of the each test, the total energy was obtained from Eq. (5) by summing the accumulated energy calculated for each cycle of a specimen. The amount of total energy is equal to the sum of the amount of dissipated and elastic recovery energies of the specimen.

The accumulated hysteretic energy absorbed by beam-column joints was numerically evaluated to study the ductility of the specimens. The amount of dissipated energy for each specimen was obtained by summing the areas in the hysteresis loops. Furthermore, to evaluate the load carrying capacity of the specimens, the elastic recovery energy by the elastic behavior of the specimens during loading was calculated. The total, dissipated, and elastic recovery energies within a loop of the load-displacement hysteresis were determined for each displacement level of the beam tip and are shown in Fig. 7.

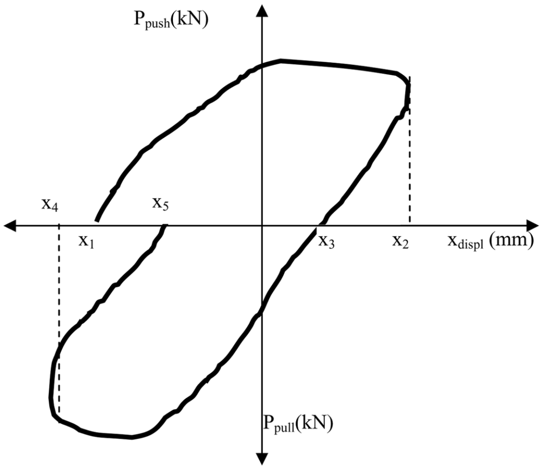

Fig. 7 Algorithm for the evaluation of energy and equivalent damping ratio for a beam tip load-displacement hysteresis loop model

6.4 Total and accumulated energy

Accumulated energy at each specimen depends on each displacement levels and the resulting reversible cyclic loads. The accumulated hysteretic energy is calculated for each peak displacement level of the beam tip force-displacement hysteresis loops. The variations in the accumulated energy versus displacement of all specimens are shown in Fig. 8 for the first cycle of each loading. As it is seen, the accumulated energy of SFRC2hstr increased as the displacement increased up to the ultimate displacement of 35 mm, whereas the accumulated energies of the other specimens

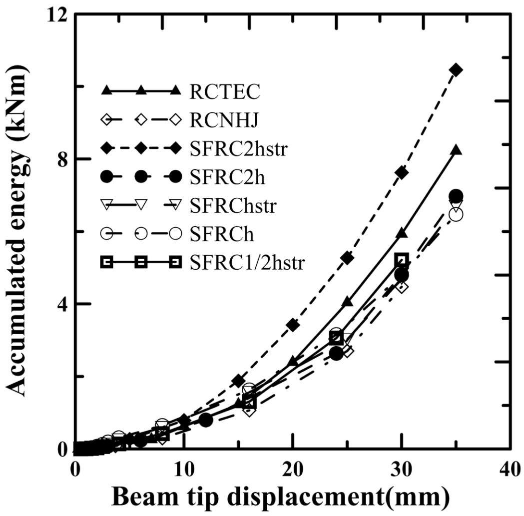

Fig. 8 Variation of the accumulated energy as a function of beam tip displacement for all specimens

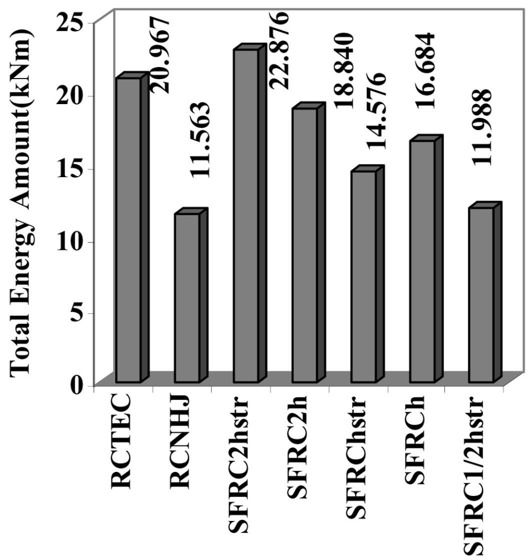

Fig. 9 Total energy capacities of the specimens

remained almost constant or decreased after the displacement of 30 mm. According to these results, the use of SFRC together with only one stirrup in the joint is more effective than hoops required for ductile RC beam-column joints by the current earthquake codes. The test results for SFRC2hstr show that the use of SFRC instead of hoops is much more effective on the ductile behavior of the exterior beam-column joints.

At the end of the tests, the amount of the total energy was obtained as the sum of the accumulated energy for each specimen at each cycle. The calculated amount of total energy for each specimen is shown in Fig. 9. The following results can be drawn from Fig. 9:

a. SFRC2hstr has more total energy capacity than other specimen. Although SFRC2hstr and SFRC2h have same properties except for one stirrup in the joint, SFRC2hstr with only one stirrup in the joint had 50% more total energy than SFRC2h. This result indicates that the total energy capacity of the beam-column joints subjected to reversible cyclic loads can be increased by using SFRC in the joint and in the confinement regions and placing at least one stirrup into joint together with SFRC.
b. RCTEC which was made of ordinary RC and detailed in accordance with TEC displayed better ductile behavior and carried larger ultimate load and had larger displacement capacity than RCNHJ having no hoops in the joint.
c. SFRCh had 2% steel fiber by volume. However, SFRChstr had 1% steel fiber by volume and one stirrup in the joint. The amount of total energy absorbed by SFRCh had 14% more than that of SFRChstr. It may seem that this trend between the specimens SFRChstr and SFRCh is insignificant. However, this result shows that such increase in fiber amount used in the beam-column joints reverses the trend of 50% between the specimens SFRC2hstr and SFRC2h. Furthermore, this result is very important to emphasize the effects of the increase in fiber content of concrete mixture on the behavior of RC beam-column joints.
d. When the amount of total energy of SFRC2hstr, SFRChstr, SFRC1/2hstr are compared to each other, SFRC2hstr had 94% and 235% more total energy than SFRChstr and SFRC1/2hstr respectively. This result shows that the extent of the region where SFRC was placed is more effective on the behavior of the beam-column joints subjected to reversible cyclic loads.

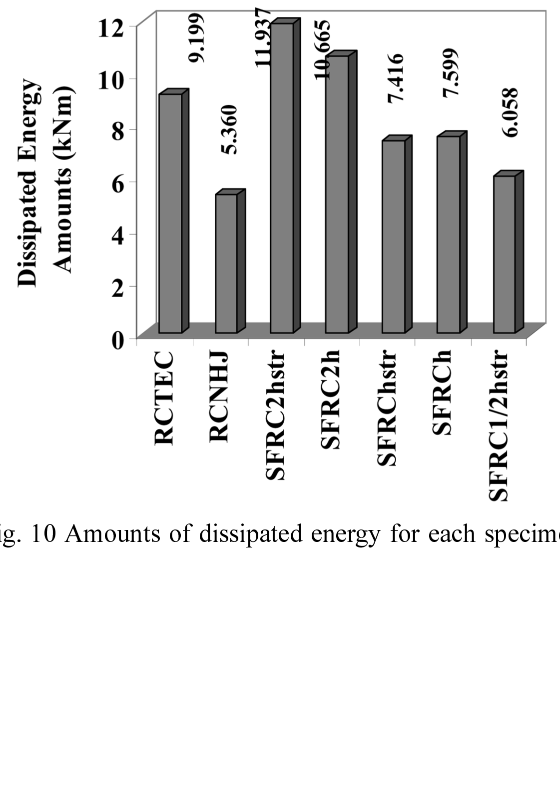

Fig. 10 Amounts of dissipated energy for each specimen

6.5 Dissipated energy

The energy dissipated in a beam-column joint through a plastic deformation is the sum of the areas in the beam tip force-displacement hysteresis loops. The amounts of the dissipated energy for all specimens are given in Fig. 10. Fig. 10 indicates that the dissipated energy amounts of SFRC2hstr and SFRC2h are higher than that of the other specimens. Ductility of a beam-column joint can be defined as the amount of dissipated energy absorbed during its plastic deformation. The test results show that the ductility of a beam-column joint can be increased by using SFRC, as well as hoops in the joint and the confinement regions of the beam and column. However, the amount of the dissipated energy for RCNHJ, which has no transverse reinforcement in the joint, is less than those of the other beam-column joint specimens, i.e., RCNHJ shows more brittle behavior than the other specimens. This result shows that the use of special transverse reinforcement in the beam-column joint is of prime importance, for obtaining ductile behavior.

6.6 Elastic recovery energy

The amount of the energy absorbed through the elastic deformation of the specimens during loading is given back to the system in the course of unloading the specimens. This energy reserved by elastic behavior is defined as the elastic recovery energy. The elastic recovery energy capacity is an indicator of the residual elasticity capability of a structural element subjected reversed cyclic load, when the reversed cyclic load is unloaded (i.e., after earthquake shock). Ductility and load carrying capacity of beam-column joint is proportional to the residual elastic capability of a structural element. The amount of the elastic recovery energy for each specimen was calculated by subtracting the amount of total dissipated energy from the amount of total energy. The elastic recovery energy capacities of the specimens are shown in Fig. 11.

Fig. 11 indicates that the elastic recovery energy capacities of RCTEC and SFRC2hstr are higher than those of the other specimens. Based on these results, the ductility and the strength of the beam-column joint specimens subjected to reversible cyclic loads can be improved by using SFRC in the joint area and in the confinement regions of the beam and columns.

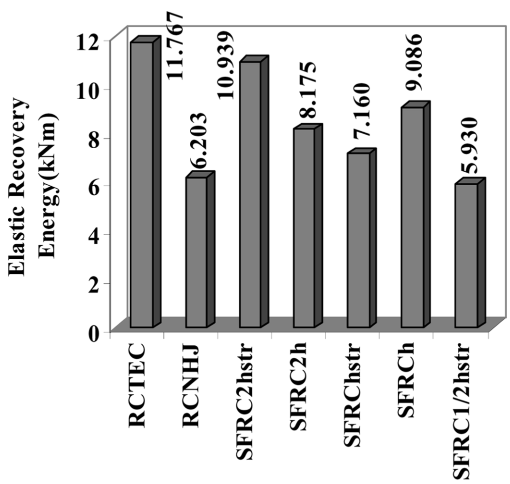

Fig. 11 Stored energy capacities of the specimens

6.7 Moment-curvature hysteretic curves and flexural rigidities of plastic hinge zone of beam

Formation of plastic hinges in columns during an earthquake should be avoided and the preferred location for plastic hinges is in the beams. Thus, beam-column joints in RC structures can safely dissipate the seismic energy by preventing global failure of the structure (Park 1986). In order to decrease the probability of plastic hinge formation in the columns, frames must be designed to have ‘strong columns and weak beams’ (TEC 1998, ACI 318-02 2002, Park 1986, Pristley and Calvi 1991). This requirement was taken into account for all of the test specimens and the total bending moment strength of the columns was 23% higher than the beam. All of the specimens failed due to widening of the bending cracks in the beam plastic hinge region. For each loading step, the shortening and elongation at the compression and tension fibers of the beam section were measured by means of transducers and the corresponding strains at these fibers were calculated. Assuming the length of the plastic hinge zone from the column face was equal to the effective depth of the beam, the average bending curvature of this zone was obtained by using the evaluated such as the curvature ϕ is obtained as

$$\phi = \frac{\varepsilon_t - \varepsilon_b}{d_b - d'} \tag{9}$$

where, ε_t and ε_b are the strains of the top and the bottom fiber of the middle section of the plastic hinge zone respectively, d_b is distance from the extreme compression fiber to tension reinforcement, d' is distance from extreme compression fiber to compression reinforcement.

$$EI = M/\phi \tag{10}$$

By using the bending moment (M) at the mid-length of the plastic hinge zone and the curvature (ϕ), the flexural rigidity of the section was evaluated at each displacement level that correspond to various loading levels. Moment-curvature hysteresis loops of various specimens are presented separately in Fig. 12(a)-(d), and the variations of flexural rigidity vs. the tip displacement are given in Fig. 13 for each specimen. Inspection of the moment-curvature hysteresis loops for each specimen revealed that the formation of the plastic hinge in the beams of SFRC1/2hstr and RCNHJ started as the cracks at this region began to widen and it fully formed by the end of the test. However, the moment-curvature hysteresis loops of the other specimens were pinched toward the origin for each cycle. Consequently, the plastic hinge did not fully develop at the plastic hinge zone of the beams in these specimens. The slopes of the ascending and descending branches of the moment-curvature hysteresis loops for RCTEC, SFRC2hstr, and SFRC2h are larger than those of the other specimens, which indicate that the flexural rigidity of these specimens is substantially higher than the remaining ones (Fig. 13 and Table 2). It is apparent that the plastic hinge zone of SFRC2hstr has the largest flexural rigidity, which indicates that the use of SFRC increases the flexural rigidity and strength significantly. However, the failure crack of SFRC1/2hstr occurred at the junction of plain concrete and SFRC in the beam (i.e., $0.5h_{beam}$ from column face). The transverse reinforcements were adequately placed into the columns and beams of the beam-column joint specimens to resist the shear forces occurred due to the reversed cyclic loads. For this reason, the failure crack of SFRC1/2hstr specimen occurred at the junction of plain concrete and SFRC in the beam due to the difference of the flexural rigidity and strength between plain concrete and

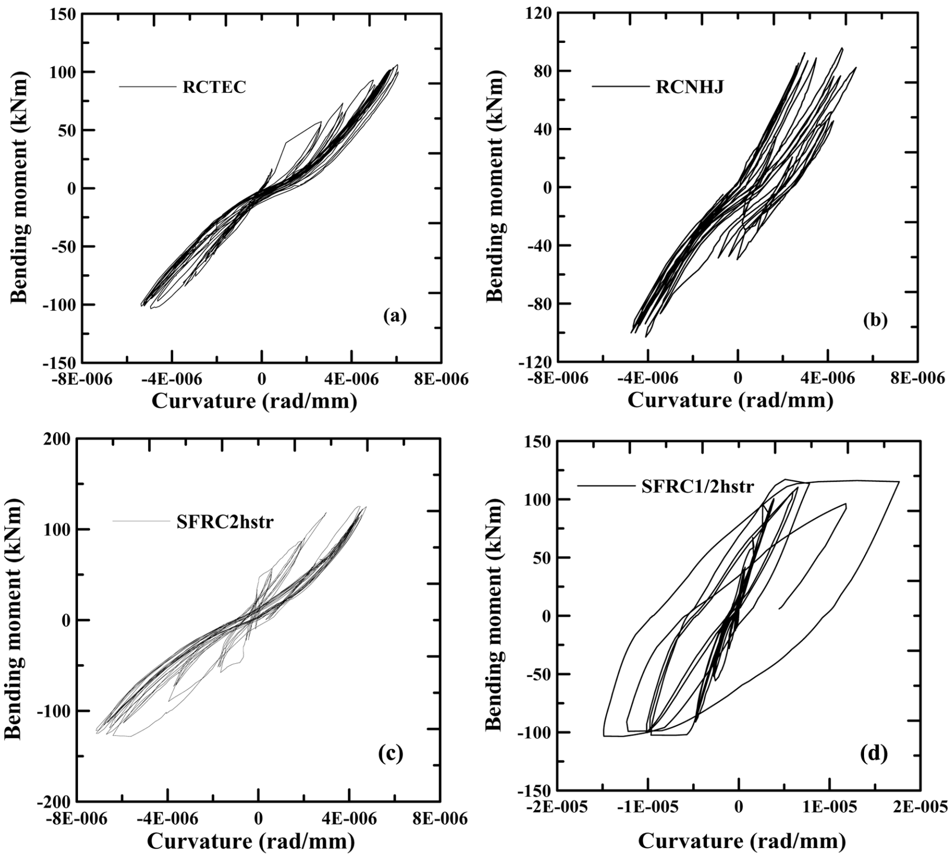

Fig. 12(a)-(d) Moment - curvature hysteresis loops of the selected specimens

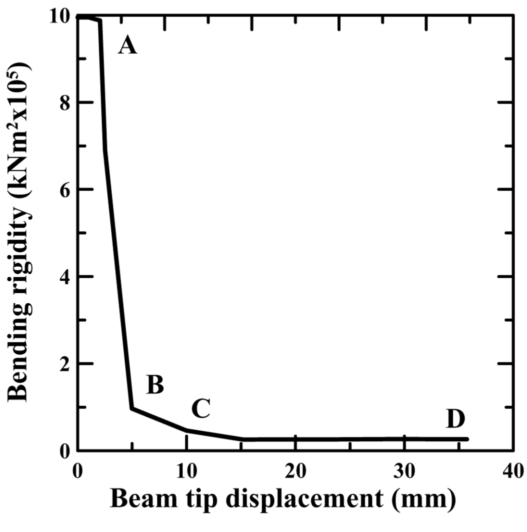

Fig. 13 Typical variation of the bending rigidity and displacement level of the beam plastic hinge zone in the beam-column joints subjected to the reversible cyclic loads

Table 2 Values of the critical points on the typical flexural rigidity – displacement curve of the plastic hinge zone of the beam in the exterior beam–column joints subjected to reversible cyclic loads

Specimen	Rigidity		and displacement at		the plastic hinge		zone	
	A		B		C		D	
	Rigidity	Disp	Rigidity	Disp	Rigidity	Disp	Rigidity	Disp
	$kNm^2 \times 10^5$	mm	$kNm^2 \times 10^5$	mm	$kNm^2 \times 10^5$	mm	$kNm^2 \times 10^5$	mm
RCTEC	5.50	0.34	0.22	6.80	0.20	9.74	0.18	35.24
RCHNJ	1.32	1.50	0.59	6.00	0.16	10.00	0.11	30.02
SFRC2hstr	9.88	2.06	0.97	4.98	0.46	9.98	0.26	35.74
SFRC2h	0.77	0.58	0.28	6.04	0.23	10.00	0.21	35.38
SFRChstr	1.39	0.54	0.37	4.02	0.21	10.00	0.15	35.94
SFRCh	0.91	0.48	0.28	6.72	0.27	10.00	0.19	35.16
SFRC1/2hstr	0.15	0.32	0.21	4.00	0.19	8.14	0.089	31.04

SFRC. This fact may lead to a recommendation that the minimum extent of the region with SFRC at the beam for external beam-column joints should be longer than $0.5h_{beam}$ from the column face for proving a smooth transfer of stresses.

7. Conclusions

Previous similar researches took into consideration the aspect ratios, volume fraction, type of steel fibers as test parameters in the use of SFRC. The results of these researches indicate that SFRC can alternatively be used instead of hoops to obtain the ductile behavior of beam-column joints. The present experimental test parameters selected in this study include the extent of the region of SFRC, the use of stirrup together with SFRC in joints, the use of hoops in the joints of RC beam-column joints and amount of steel fibers added into concrete mixtures.

The test results were evaluated in terms of the load carrying capacity, the shear strength, the ductility, and the amounts of the total, the dissipated, the elastic recovery energies of the external beam-column joint specimens, and the flexural rigidities of plastic hinge regions of the beams in these assemblages. The ductility of structures and structural elements directly depends on their displacement capability of them and the amounts of total energy absorbed by them also depend on displacements in those. Because of that, in the present paper, the ductility of beam-column joint specimens subjected to reversed cyclic loads is evaluated according to the total energy amounts of specimens.

The results which are comprehensively discussed in each of the evaluation chapters are briefly presented, as follows:

a. Changes in the extent of the region placed SFRC is significantly effective on the behavior of beam-column joints.

b. Use of stirrup together with SFRC in the joints increases the cyclic load capacity, the ductility and the amounts of total absorbed energy of beam-column joints.

c. As the volume fraction of steel fibers added into concrete mixture is increased up to 2% by volume, the strength and the total energy amount of the beam-column joint specimen without stirrup in the joint comes close to the behavior of beam-column joint with stirrup in the joint.

d. To prevent early failure of beams in assemblages, the extent of the region of SFRC used in the beam of beam-column assemblages should be longer than $0.5h_{beam}$.
e. The use of SFRC can reduce the difficulties associated with placing the transverse and longitudinal reinforcements in the confinement regions of the beam-column joints due to eliminating the hoops. Thus, SFRC can be accepted as a potential alternative to hoops in beams, columns and joint of beam-column assemblages.

However, it may be stated that further research is required to evaluate the effects of the different volume fraction of steel fibers at various parts of the beam and column on the rigidity, ductility, strength and total energy amount of beam-column joints.

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