Bi-directional seismic behavior of steel beam-column connections with outer annular stiffener

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DOI:

[10.1016/j.engstruct.2020.111443](https://doi.org/10.1016/j.engstruct.2020.111443)

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Document Version

Peer reviewed version

Citation for published version (Harvard):

Bai, Y, Wang, S, Mou, B, Wang, Y & Skalomenos, KA 2021, 'Bi-directional seismic behavior of steel beam-column connections with outer annular stiffener', *Engineering Structures*, vol. 227, 111443.
<https://doi.org/10.1016/j.engstruct.2020.111443>

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Experimental study of steel beam-column connections with external diaphragm subjected to bi-directional cyclic loadings

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Abstract: This paper investigates the seismic behavior of steel beam-column connections with external diaphragm under bi-directional cyclic loadings. Five welded three-dimensional beam-column connections configured by hollow square structural steel (HSS) column sections, H-shaped steel beams, and external ring stiffener were tested. The main parameters considered in this experimental study include loading protocols (anti-symmetric bidirectional and unidirectional cyclic reversals), beam depths, and the leg size of fillet weld between the external ring stiffener and column. Moment versus rotation hysteretic responses, skeleton curves, stiffness degradation, and the associated energy-dissipating capacity were analyzed. The hysteretic loops of all specimens show a stable and ductile performance indicating a good energy-dissipating capacity. The ultimate flexural strength under the uni-directional cyclic reversals was found to be 20% higher than that under the corresponding bi-directional loading history. As the fillet weld leg size increases from 6 mm to 10.2 mm, the ultimate flexural capacity of the beam ends is slightly enhanced. The beam depth is a major factor that controls the elastic stiffness and strength capacity of the external ring stiffener. The ductile failure mode can be achieved by using the proposed equations for the strength calculation of ring stiffeners.

Keywords: Steel beam-to-column connection; Bi-directional loading; Beam depth; Fillet weld size; External ring stiffener; Cyclic behavior

28 1. Introduction

29 Since the Northridge earthquake (1994) and Hyogoken-Nanbu Earthquake (1995),
30 investigations on seismic performance of steel moment-resisting frame (SMRF) buildings
31 revealed brittle failure mechanisms in welded beam-to-column connections (Kuwamura 1998;
32 Mahin 1998; Miller 1998; and Nakashima et al. 1998; Joh and Chen 1999; and Suita et al.
33 2008), and excessive side-sway vibrations were observed in high-rises SMRFs during the
34 Tohoku earthquake (2011). Brittle damages have been observed at the welded portions
35 between the lower flange of beams and column flanges, and the associated stiffness
36 deterioration tends to reduce the earthquake-induced collapse capacity of low-rise and
37 high-rise SMRFs (Khandelwal and El-Tawil 2007; Lignos and Krawinkler 2011; Takewaki et
38 al. 2011; Lignos et al. 2013; and Bai et al. 2015).

39 The fabrication of steel moment connections between beam and column should apply
40 various types of diaphragms with internal, external, or through ring stiffeners. The type of
41 diaphragm plays a critical role in the force transfer and distribution of bending moment from
42 beam to column components. Extensive experimental studies have been carried out on
43 connections with external diaphragms (Yoshizato et al. 1994; Park et al. 2005; Li et al. 2010;
44 Vulcu et al. 2017; and Rong et al. 2019), internal diaphragms (Cheng and Chung 2003; Chou
45 and Jao 2010; Qin et al. 2014; and Kang et al. 2015), and through diaphragms (Qin et al.
46 2014; Cao et al. 2018; and Yu et al. 2019). Nonetheless, most of the aforementioned studies
47 are only focused on the performance of plane connections and ignore the interaction of
48 bi-directional forces at beam ends that do exist in actual buildings. Jiao et al. (2011) have
49 found that the directions of loading history significantly influences the strength and
50 deformation capacity of steel moment connections. In engineering practice, the design of
51 panel zones (PZ) in SMRFs should account for the three-dimensional beam-column
52 connections. Moreover, Uang et al. (2000) and Civjan et al. (2000) investigated the cyclic

behavior of pre-Northridge welded steel moment connections, and they found that brittle fracture in the groove weld on the top of beam flange could not be prevented unless new weld metal was introduced.

In order to enhance the seismic performance of steel moment connections, new design details have been developed based on the simple idea that the plastic hinge should be formed far away from the PZ to avoid its premature shear failure prior to the development of the full plastic strength of beams (Fukumoto and Morita 2005; Fülöp and Dubina 2006; Chen et al. 2017; Rong et al. 2018; and Mou et al. 2019). In addition, to avoid brittle fracture at connecting parts and improve the deformation capacity, beam-to-column connections with external ring stiffener (diaphragm) are preferably used to high-rise steel buildings after Hyogo Ken Nanbu earthquake (1995). For instance, Lai et al. 2019 have established a database of moment connections in composite special moment frames, and various new types of steel connections were developed to enhance seismic performance (Ricles et al. 2002; Kwon et al. 2006; and Wang et al. 2015).

This paper investigates experimentally the cyclic behavior of three-dimensional steel beam-to-column connections located at the corner of high-rise SMRFs. Anti-symmetrically bidirectional cyclic loadings were applied on both beam ends with incremental displacement amplitudes to investigate the deformation capacity and stiffness degradation. The connection detail consists of a square hollow steel section (HSS) column welded through an external ring stiffener to H-shaped steel beams. The steel material Q390 was adopted for specimen fabrication. Five specimens were tested to evaluate the effects of beam depth, loading pattern, external ring stiffener, and the effective thickness of fillet weld on hysteresis behavior and failure modes. Equations are finally provided to determine the shear resistance of PZs, and their efficiency is verified using the experimental results.

2. Experimental program

2.1 Design of test specimens

Five half-scale three-dimensional H-shaped steel beam-to-square hollow steel sectional (HSS) column connections were designed and fabricated with various panel shapes and component dimensions as summarized in Table 1. The connection specimens had identical steel materials (Q390) and dimensions of HSS columns (square tubular section: 200×200×9 with compact slenderness ratio D/t of 22). For the H-shaped steel beams, various geometries were adopted to evaluate the influences of web depth (200, 300, and 450mm). Particularly, the specimen No. 1 was designed as the reference specimen and was a three-dimensional beam-to-column connection with beam subjected to bi-directional cyclic loading. The results are compared with those of the identical specimen No. 2 which was the corresponding planar beam-to-column connection subjected to uni-directional cyclic loadings. The influences of the beam depth (No. 3 and 4) and the fillet weld leg size (No. 5) between the HSS column and external ring stiffener on the ultimate flexural capacity of connection specimens were evaluated as well. Fig. 1 shows the plan and elevation views of the reference specimen No. 1. The total length of the square HSS column was 1626 mm including height for the panel zone equal to 276 mm, and the beam lengths were equal to 1500 mm in the south and west directions respectively. The beam components were connected to each other in 90° and parallel to the ground to build up a corner connection of high-rise SMRFs with column-to-beam over-strength ratio equal to 1.5. The external ring stiffeners (i.e., diaphragms) were designed to have a polygon shape for optimizing the force transfer and distribution along the 45° edge of diaphragms. The diaphragms were connected to beam flanges using the full penetration butt weld, and fillet welded around the perimeter through the out surface of the square HSS column. The thickness of the weld seam is 9 mm with 35° of the groove at the beam side. The external ring stiffeners with a thickness of 12 mm were split into two plates having a 2 mm seam as shown in Fig. 2.

The material properties of structural steels were determined by conducting tensile coupon tests. The coupon was deducted from the external diaphragm, the beam flange and web, and the square HSS column. Table 2 summarizes Young's modulus, yield stress, ultimate tensile stress, and elongation ratio measured in the tensile tests of steel coupons with various thicknesses.

2.2 Test setup and loading protocol

Fig. 3 illustrates the test setup for the three-dimensional connection specimens. Two MTS hydraulic servo actuators were installed vertically on the two beam ends to apply cyclic loading with anti-symmetrical amplitudes. The distance between the central line of column and loading point is 1500 mm which is treated as the effective length of beam components. The height between upper and lower column boundaries is equal to 1626 mm. The upper and lower boundary supporters are fixed with the foundation and reaction frame to reduce the buckling length of the column for the purpose of this experimental study aiming to investigate the cyclic behavior external diaphragm, shear deformation capacity of panel zones, and flexural strength of beam ends. Furthermore, in order to enhance the lateral stiffness of test facilities, a braced out-of-plane restrainer was placed between the column base and the top of the reaction frame.

The loading was controlled by imposing quasi-statically the cyclic displacements with the incremental amplitudes. Fig. 4 shows the mechanism of the bi-directional cyclic loading corresponding to the story drift angle of 0.2%, 0.5%, 1%, 2%, 3%, and 4%. Two cyclic reversals were repeated for each level of amplitude. The loading ended when the displacement reached the maximum capacity of the actuator in a positive direction. Anti-symmetrically vertical displacements were imposed by the actuators on the west and south beams. It means that if the west beam sustains a positive rotation angle, the equivalent

negative amplitude will be applied to the south one and vice versa. Fig. 4(b) shows the loading history which was anti-symmetrically applied on both south and west beams.

2.3 Measurements

Displacement transducers were placed to measure the local deformations such as horizontal displacements of column and panel zone, vertical displacements at beam ends, and external ring stiffener, as shown in Fig. 5. Vertical forces at both loading points were recorded by the sensors in hydraulic actuators, so the bending moment at beam ends $M = PL$, where P is the vertical forces and L is the effective length of beam components between the loading point and the central line of the column web (1500 mm). Then, the rotational angles at beam ends can be calculated as follows:

$$\theta_{blw} = \frac{V_1 - V_w}{L - D/2} - \frac{U_{wl} - U_{wu}}{d_b} \quad (1)$$

$$\theta_{bls} = \frac{V_2 - V_s}{L - D/2} - \frac{U_{sl} - U_{su}}{d_b} \quad (2)$$

where D is the depth of columns, and d_b is the height of the beam.

3. Discussion of test results

3.1 Hysteretic moment-rotation responses

The hysteresis curves between bending moment and rotational angle of connection specimens show the maximum strength capacity and ultimate deformation capacity as critical indices to quantify the seismic behavior of specimens. Fig. 6 shows the hysteresis curves at each beam end of the test specimens. The maximum bending moments in positive and negative directions are highlighted as solid dots. Nearly no strength and stiffness deterioration were observed when the specimens achieved a maximum rotation angle of 0.04 rad, but cyclic strain hardening slightly increased the bending moment at each level of amplitudes.

3.2 Major failure modes

The limit states of the test specimens are considered the yield and full-plastic points calculated at $1/3 K_e$ and $1/6 K_e$, respectively, where K_e is the elastic flexural stiffness determined by a secant line corresponding to a story drift angle (SDR) equal to 0.2%. Fig. 7 illustrates the various failure modes of each connection specimen.

The yielding state of the test specimen No. 1 was observed in the second loading cycle of $SDR = 2\%$. Thereafter, when the maximum SDR exceeds 2%, out-of-plane local deformation occurred in the external ring stiffener, which resulted in a compressive force and inward local buckling on the column face. The deterioration of the connection strength capacity was initiated after the occurrence of fracture at the external ring stiffener, slightly prior to the completion of loading. The specimen No. 2 yielded in a similar SDR with specimen No. 1, but it reached 13% larger maximum flexural strength due to the unidirectional cyclic loading. The bi-directional loading leads to a certain reduction of strength capacity, indicating an overestimation on the seismic capacity of beam-to-column connections determined based on unidirectional loading.

Furthermore, the Specimen No. 3 with smaller beam depth (200 mm) yielded at the second loading cycle of $SDR = 2\%$, but the ring stiffener deformation and column buckling were not evident compared with those of the Specimen No. 1. The maximum beam moment was developed at $SDR = 4\%$, and no fracture occurred in the connections between the column and external ring stiffener. On the other hand, the increase of the beam depth in Specimen No. 4 with the beam depth of 450 mm resulted in a significantly greater maximum flexural capacity by 52%, while the corresponding story drift ratio demonstrated the smaller ductility with Specimen No. 1. During the second loading cycle of $SDR = 4\%$, a fracture occurred in the external ring stiffener at the bottom flange of the west beam, and propagated along the diagonal direction until ultimate failure, as shown in Fig. 7(h). This indicates that the ring

stiffener sustained a significant amount of tensile forces along the symmetrical line between both beams.

The yielding of specimen No. 5 (minimal size of weld leg w : 6 mm) occurred during the second loading cycle of $SDR = 1\%$, earlier than that of the specimen No.1 which was designed with a larger weld leg size $w = 10.2$ mm. Then, inward local buckling occurred in the panel zone of HSS column due to the compression loads transferred from the external ring stiffener. As shown in Fig. 7(j), the brittle fracture was initiated across the weld seam between the external ring stiffener and column after the loading of $SDR = 3\%$, but the flexural strength increased to the maximum capacity at the second loading cycle of $SDR = 4\%$.

3.3 Ultimate flexural capacity

Fig. 8 shows the skeleton curves of the hysteresis loops by connecting the peak points of each cycle. Similar elastic stiffness and flexural strength capacity were observed in both of the west and south beam components of the test specimens No. 1, 2 and 4 under antisymmetric cyclic loads, [see Fig. 8(a), Fig. 8(b), and Fig. 8(d)]. In comparison with the specimen No. 4 with the largest beam depth 450 mm, the variation in flexural strengths of both beams was triggered when the rotation increases to 0.02 rad, as shown in Fig. 8(c). The strength reduction of the west beam was caused by brittle fracture initiated and propagated across the weld seam between the lower flange of the west beam and external ring stiffener.

The skeleton curves of the specimens with various beam depths are presented in Fig. 9, which is noted that the beam depth plays significant effects on the elastic stiffness and ultimate flexural strength of connection assembly. For instance, the elastic stiffness and ultimate flexural strength of the connection with beam-depth of 300 mm (No. 1) increased by 110.9% and 66.9% in the positive direction, and 117.6% and 66.4% in the negative direction compared to that with 200 mm in beam depth (No. 3), respectively. Moreover, compared with the connection specimen with beam depth 300 mm, the elastic stiffness and ultimate flexural strength of the specimen No. 4 (beam depth: 450 mm) were further increased by 139.7% and

55.5% in a positive direction, and 140.6% and 54.8% in the negative direction. This proportional tendency between the elastic stiffness

Fig. 10 compares the influence of loading mode on the skeleton response curve of the connections. It is noted that the elastic stiffness and flexural strength capacity of specimen No. 1 are 18.0% and 34.8%, and 18.8% and 15.9% lower than those of specimen No. 2 respectively in positive and negative directions. This is because the elastic flexural stiffness of specimen No. 1 was correspondingly weakened by the torsional deformations due to bi-directional loading protocol. Furthermore, the influences of the fillet weld leg size on the ultimate strength and deformation capacity were evaluated by comparing the skeleton curves of the specimen No. 1 with that of the specimen No. 5, as shown in Fig. 11. One can observe that using 10 mm as the fillet weld leg size would improve the elastic stiffness and maximum strength capacity by 9.2% and 3.6%, and 21.1% and 11.8% respectively in positive and negative directions.

Table 3 accordingly summarizes the values of important design quantities such as the elastic stiffness K_e , the yielding moment M_y , yielding rotational angle θ_y , full-plastic moment M_p , full-plastic rotational angle θ_p , and maximum moment capacity $M_{\max}$. It is noted that the depth of the beam cross-section significantly increase the yielding strength and full-plastic flexural strength, and similar influences of uni-directional loading and weld leg size were found as well. Nonetheless, brittle fracture were observed in the external ring stiffener of the specimen No. 4 with 450 mm in beam depth. This phenomenon implies the significance of ring stiffener design to satisfy the design demand of connection ductility.

3.4 Stiffness deterioration

In addition to the elastic stiffness and ultimate flexural strength capacity, the deterioration in unloading stiffness quantifies the extent of inelastic damages caused by the accumulation of plastic deformations subjected to cyclic reversals and can be estimated as follows:

$$K_j = \sum_{i=1}^2 M_{i,j} / \sum_{i=1}^2 \theta_{i,j} \quad (5)$$

where K_j is the reloading stiffness of the j -th cyclic amplitude, $M_{i,j}$ is the maximum bending moment at the i -th reversal of the j -th cyclic amplitude, and $\theta_{i,j}$ is the associated rotational angle (rad). Fig. 13 compares the relationship between the reloading stiffness versus the number of loading reversals. Overall, the beam depth demonstrates the most significant impact on stiffness degradation compared with the other influential parameters. The degraded stiffness reduces after the specimens achieved the maximum $\text{SDR} = 2\%$. Table 4 presents the detailed values of the stiffness ratios K_p/K_y and $K_{0.04}/K_y$, where K_y , K_p , $K_{0.04\text{rad}}$ are respectively the unloading stiffness corresponding to yielding, full plasticity and $\text{SDR} = 4\%$. Results indicate that the average $K_{0.04}/K_y$ value of the specimen No. 1 is 0.565 which is slightly smaller than that of the specimen No. 2 (0.595). On the contrary, the specimen No. 4 exhibited a notable stiffness degradation due to the large beam depth (450 mm) which sustained a large amount of the plastic moment that suddenly lost after the occurrence of fracture through the external ring stiffener. The results indicate that large column-over beam strength ratio could be effective to reduce the stiffness degradation due to a potential fracture in the external diaphragm.

The equivalent viscous damping ratio ξ was also computed to evaluate the energy dissipating capacity of each specimen. For the hysteresis loops shown in Fig. 14, the value ξ can be obtained by (Mou and Bai 2018) as follows:

$$\xi = \frac{1}{2\pi} \frac{S_{ABC} + S_{CDA}}{S_{OBE} + S_{ODF}} \quad (6)$$

where $S_{ABC} + S_{CDA}$ indicates the area of hysteresis loops in on cycle (14), and $S_{OBE} + S_{ODE}$ indicates the area of the corresponding triangle as given in Fig. 14. It can be found in Fig. 15 that the increase of energy dissipation was relatively slight prior to story drift ratio = 0.01 rad, but thereafter the accumulation of plastic energy progressively increases. Specimens with

greater beam depth demonstrated higher energy dissipating capacity, while the fillet weld leg size showed minor influences.

A similar tendency of the equivalent viscous damping ratio with respect to the amplitude of rotational angle was obtained, as illustrated in Fig. 16. Characteristic values are summarized in Table 5, where $\zeta_{eq,y}$, $\zeta_{eq,p}$, and $\zeta_{eq,0.04rad}$ are respectively the equivalent viscous damping ratios at yielding, full plasticity and story drift ratio = 0.04 rad.

4. Shear resistance of panel zones

4.1 Strain distribution of external diaphragm

Fig. 17 shows the location of the strain gauges attached on the upper and lower surfaces of the external ring stiffener. Uniaxial strain gauges were placed along the longitudinal direction of west and south beams to measure the forces transferred from the bending moment at the beam ends. A tri-axial strain gauge was placed on the symmetry axis of the two beams in order to measure the deformation on the external ring stiffener due to bi-directional loading. Fig. 18 shows the strain distributions of the test specimens, in which the horizontal and vertical axes indicate the strain responses in west and south directions respectively. The values in elasticity, yielding and full plasticity are respectively marked by the shapes of hollow circle, trilateral and quadrilateral. In general, the strain values proportionally increase when the cyclic loading history was incrementally applied, as seen in Fig. 18(b). However, the external ring stiffener under bi-directional loading yielded at the location close to the diagonal line between west and south beams and the other parts remain in elasticity. For this reason, a stress concentration near the corner of HSS columns was formed and the external stiffener ring in this area was likely to experience rupture due to excessive tension force. Furthermore, the ring stiffeners in the west and south sides completely yielded when the specimen reached its full plasticity.

4.2 Shear deformation of panel zones

The shear deformation angle is a critical index to evaluate the shear resistance of panel zones. Horizontal and vertical displacements (PH_u , PH_l , PV_r , and PV_l) were measured by the associated transducers given in Fig. 19 to calculate the deformation angle based on the Eq. (3) as follows:

$$\gamma = \frac{PH_u - PH_l}{h_p} - \frac{PV_l - PV_r}{b_p} \quad (7)$$

where h_p is the vertical height of the joint, b_p is the horizontal width of the panel zone, PH_u is the horizontal displacement at the upper center point of panel zones, PH_l is the horizontal displacement at the lower center point of panel zones, PV_l is the vertical displacement at the left center point of panel zones, and PV_r is the vertical displacement at the right center point of panel zones.

Fig. 20 shows the maximum shear deformation angle in panel zone when subjected to each level of cyclic reversals. Generally, the maximum shear deformation angles of the panel zone of the south beam were 20%~50% greater than those of the west beam. It was due to the fact that the elastic stiffness of the south beam is lower than the west beam which leads to evident torsions occurred in the south beam. More specifically, shear deformations of the two-panel zones of specimen No. 1 were about 18%~29% lower than those of specimen No. 2 which was subjected to uni-directional cyclic loading, as compared in Figs. 20 (a)-(b). The shear deformation in panels also increases proportionally to the beam depth as shown in Figs. 20 (a), (c) and (d). The ratios of the shear deformation relative to the total deformation were found to be 8%, 25%, and 36% respectively for the specimens No., 3 (200 mm), No. 1 (300 mm) and No. 4 (450 mm). In addition, the shear strain response in panel zones propagates along with the increase of story drift angle as given in Fig. 21, where the shear strain was calculated by the measured values in the horizontal, vertical and diagonal directions according to the following equation:

$$\gamma_{xy} = 2\varepsilon_3 - (\varepsilon_2 - \varepsilon_1) \quad (8)$$

where ε_1 is a vertical strain, ε_2 is a horizontal strain, and ε_3 is 45° diagonal strain.

5. Design of External Ring Stiffeners

In order to calculate the bending moment of connections, the yield failure mechanism of the external diaphragm needs to be clarified. The tension and compression strengths of the external diaphragm (P_p) would induce horizontal tension forces transferred to beam flanges through the external diaphragm, as presented in Fig. 22(a). The tension force P_y sustained by the external diaphragm can be calculated as follows:

$$P_y = \left[\frac{4(x+t/2)}{\sqrt{3\{1+a^2/4(x+t/2-b)^2\}}} + \frac{4(1+\tan\theta)h_d}{\sqrt{3(1+4\tan^2\theta)}} \right] t_d \sigma_{dy} \quad (9)$$

where x is the horizontal length of the plastic area, t is the thickness of HSS column, t_d is the thickness of beam flange, h_d is the distance between column surface and external diaphragm, θ is the angle between diaphragm hypotenuse and horizontal line, and σ_{dy} is the yield stress of the external diaphragm. The variable x can be solved by combining the following Eqs. (10) and (11):

$$\frac{t_d \sigma_{dy} (x+t/2-b)}{\sqrt{12(x+t/2-b)^2 + 3a^2}} - \frac{t^2 \sigma_{dy} \{t_d + 2s + (D-t)/\kappa\}}{4x^2} = 0 \quad (10)$$

$$2(\pi + 4\kappa \log_e \kappa)x - (D-t)\pi = 0 \quad (11)$$

where κ is a ratio between vertical length to horizontal length of the plastic area in the panel zone, a is the distance from column surface to beam section, b is the distance from the edge of beam flange to the center of column web, s indicates the leg of fillet weld, and D is the width of HSS column. When the external diaphragm reaches its full plastic force, the associate force P_p can be derived from the conservation of energy between the plastic areas I and II (see Fig. 22), and column as presented in Eqs. (12)~(13) below:

$${}_I W_d = 2\sqrt{\frac{1}{3}} \{ (x+t/2-b)^2 + a^2/4 \} \cdot t_d \sigma_{dy} \delta \quad (12)$$

$$_{II}W_d = \frac{2(1+\tan\theta)}{\sqrt{3(1+4\tan^2\theta)}} h_d t_d \sigma_{dy} \delta \quad (13)$$

$$W_c = \left\{ \frac{t_d + 2s}{x} + \frac{D-t}{\kappa x} - \frac{2}{\kappa} + \frac{4}{\pi} (\log_e \kappa)^2 + \pi \right\} t^2 \sigma_{cy} \delta \quad (14)$$

where $_{I}W_d$ indicates the virtual work in a plastic area I, $_{II}W_d$ indicates the virtual work in the plastic area II, W_c is the virtual work acted in the column flange, σ_{cy} is the yield stress of the column, and σ_{dy} is the yield stress of the external diaphragm. The upper bound value of P_p can be derived by solving the partial differential equations with respect to x and κ as follows:

$$\begin{cases} \partial P_p / \partial x = 0 \\ \partial P_p / \partial \kappa = 0 \end{cases} \quad (15)$$

Then, the maximum P_p can be calculated in terms of the virtual work principle by substituting Eq. (15) into the follow equation:

$$P_p = (_{I}W_d + _{II}W_d + W_c) / \delta \quad (16)$$

where δ is the virtual displacement. In contrast to the external diaphragm subjected to unidirectional loading, the failure mode in case of bidirectional loadings is different and its yield strength P_y can be calculated according to the similar method by solving x the following equations:

$$\frac{t_d \sigma_{dy} (x + t/2 - b)}{\sqrt{12(x + t/2 - b)^2 + 3a^2}} - \frac{t^2 \sigma_{cy} \{3(t_d + 2s) + (D-t)/\kappa\}}{16x^2} = 0 \quad (17)$$

$$(2\pi - \kappa + 8\kappa \log_e \kappa)x - (D-t)\pi = 0 \quad (18)$$

With regards to the associated full-plastic strength P_p , the virtual works in the plastic areas I and II are calculated similarly to the one under unidirectional loading only considering two plastic zones. The virtual work in the column flange should eliminate the corner where should maintain a right angle due to the forces from bidirectional loadings, as presented in Eq. (19).

$$W_c = \left\{ \frac{3(t_d + 2s)}{4x} + \frac{D-t}{\kappa x} - \frac{2}{\kappa} + \frac{4}{\pi} (\log_e \kappa)^2 + \pi \right\} \cdot t^2 \sigma_{cy} \delta - \frac{1}{\pi} \log_e \kappa \cdot t^2 \sigma_{cy} \delta \quad (19)$$

Then, the Eq. (19) solves the W_c in tri-dimension so that the corresponding P_p can be obtained by Eq. (16). Table 3 presents the calculated results of the critical bending moments at beam ends in terms of the above design method, compared with those of the experimental values. Overall, the calculated moment values obtained by the equations were 88% to 103% of the test results. The average value of the yielding moment ratio was 0.90. In addition, the standard deviation and the variable coefficient were 0.0142 and 0.0159, respectively. The average value of the yielding moment ratio was 0.89. In addition, the standard deviation and the variable coefficient were 0.0511 and 0.0549, respectively. The proposed method works with a fairly good accuracy.

6. Conclusions

This paper investigates experimentally the cyclic behavior and deformation capacity of three-dimensional steel beam-to-column connections under bidirectional cyclic loading. The specimens are configured by square HSS columns and H-shaped beams connected together external ring stiffeners. The main parameters accounted in this series of tests are loading protocols (anti-symmetric bidirectional and unidirectional cyclic reversals), beam depths, and the fillet weld leg size between the external stiffener ring and square HSS column. Experimental results are discussed on moment versus rotation hysteretic responses, failure modes, stiffness degradation, and the energy dissipating capacity. A theoretical design method is proposed for predicting the strengths of external ring stiffeners to estimate the critical bending moment at beam ends with ductility failure mode. The following concluding remarks can be drawn:

- Overall, the tests of three-dimensional beam-to-column connections were well conducted to show similar hysteretic responses between the both perpendicular beams. It was found that the bi-directional cyclic reversals reduced the maximum flexural strength capacity nearly by 20% compared with that under the uni-directional loading

history indicating an overestimation in the design codes based on planar design equations.

- Column over-strength ratio directly controlled by the beam depth significantly affects the failure modes of connections. Large beam depth increases the flexural stiffness and strength but would lead to a premature brittle failure such as fracture in the external ring stiffener due to high tensile forces transferred from beam flanges.
- A stress concentration along the diagonal line of HSS columns occurred in three-dimensional connections and the external ring stiffener in this area is vulnerable to prevent brittle fracture due to excessive tension forces. The ring stiffeners in west and south sides of connections completely yielded when beams reached their full plasticity which is required for the critical bending moment at beam ends with satisfaction of ductility failure mode.
- The yield failure mechanism of the external diaphragm is clarified to theoretically propose a design method based on virtual work principle for predicting its yield and full plastic strengths in three-dimensional beam-to-column connections. The calculated results of the critical bending moments at beam ends in terms of the proposed design method are compared with those of the experimental values and demonstrate a fairly good accuracy.

Data Availability Statement

Some or all data, models, or code generated or used during the study are available in a repository online in accordance with funder data retention policies (provide full citations that include URLs or DOIs).

Acknowledgment

This study was financially supported by National key research and development program (2016YFC0701204), Research Award Fund for Young and Middle-aged Scientists in Shandong Province (ZR2016EEB38), Innovative funding for Postdoctoral researchers in

Shandong Province, (201601017), and Qingdao Applied Research Fund (2016194). The corresponding author would like to appreciate the sponsorship from Alexander von Humboldt Stiftung/Foundation (1196752).

References:

Bai, Y., Shi, Y., Deng, K. (2016). Collapse analysis of high-rise steel moment frames incorporating deterioration effects of column axial force–bending moment interaction. *Eng. Struct.*, 127, 402–415.

Cao, S., Shu, G., Lin, K., and Qin, Y. (2018). Experimental seismic behaviour of bottom-through-diaphragm and top-ring connection to SST columns. *Journal of Constructional Steel Research*, 150: 249-260.

Chen, C., Chung, L. (2003). Seismic performance of steel beams to concrete-filled steel tubular column connections. *Journal of Constructional Steel Research*, 59(3): 405-426.

Chen, Y., Pan, L. and Jia, L. (2017). Post-buckling ductile fracture analysis of panel zones in welded steel beam-to-column connections. *Journal of Constructional Steel Research*, 132: 117-129.

Chou, C. and Jao, C. (2010). Seismic rehabilitation of welded steel beam-to-box column connections utilizing internal flange stiffeners. *Earthquake Spectra*, 26(4): 927-950.

Civjan, S.A., Englhardt, M.D., Gross, J.L. Retrofit of pre-northridge moment resisting connections, *J. Struct. Eng.*, 2000, 126(4): 445-452.

Fülöp, L. and Dubina, D. (2006). Design criteria for seam and sheeting-to-framing connections of cold-formed steel shear panels. *J. Struct. Eng.*, 132(4).

Jiao, Y., Yamada, S., Kishiki, S., Shimada, Y. (2011). Evaluation of plastic energy dissipation capacity of steel beams suffering ductile fracture under various loading histories. *Earthquake Engineering and Structural Dynamics*, 40(14): 1553–1570.

Joh, C., Chen, W F. (1999). Fracture strength of welded flange-bolted web connections. *J. Struct. Eng.*, 125(5): 565-571.

420 Kang, L., Leon, R., Lu, X. (2015). Shear strength analyses of internal diaphragm connections
421 to CFT columns. *Steel and Composite Structures* 18(5):1083-1101.

422 Khandelwal, K. and El-Tawil, S. (2007). Collapse behavior of steel special moment resisting
423 frame connections. *J. Struct. Eng.*, 133: 646-655.

424 Kuwamura, H. (1998). Fracture of steel during an earthquake—State-of-the-art in Japan. *Eng.*
425 *Struct.*, 20(4–6): 310–322.

426 Kwon, Y. B., Chung, H. S., and Kim, G. D. (2006). Experiments of cold-formed steel
427 connections and portal frames. *J. Struct. Eng.*, 132(4).

428 Lai, Z., Fischer, E., and Varma, A. (2019). Database and review of beam-to-column
429 connections for seismic design of composite special moment frames. *J. Struct. Eng.*,
430 145(5).

431 Li, W., Chen, Y., Wang, W., Xu, Y., Lv, X. (2010). Experimental study of external diaphragm
432 joint connecting CHS column and H-shaped beam. *Advanced Steel Construction* 6(1):
433 578-588.

434 Lignos, D., Krawinkler, H. (2011). Deterioration modeling of steel components in support of
435 collapse prediction of steel moment frames under earthquake loading. *J Struct Eng*, 137
436 (11), 1291-1302.

437 Lignos, D., Hikino, T., Matsuoka, Y., Nakashima, M. (2013). Collapse assessment of steel
438 moment frames based on E-Defense full-scale shake table collapse tests. *J. Struct. Eng.*,
439 139 (1) (2013), pp. 120-132.

440 Mahin, S. A. (1998). Lessons from damage to steel buildings during the Northridge earthquake.
441 *Eng. Struct.*, 20(4–6), 261–270.

442 Miller, D. K. (1998). Lessons learned from the Northridge earthquake. *Eng. Struct.*, 20(4–6),
443 249–260.

444 Mou, B., Li, X., Bai, Y., and Wang, L. (2019). Shear behavior of panel zones in steel
 445 beam-to-column connections with unequal depth of outer annular stiffener. *J. Struct. Eng.*,
 446 145(2).

447 Mou, B., Bai, Y. Experimental investigation on shear behavior of steel beam-to-CFST column
 448 connections with irregular panel zone, *Eng. Struct.*, 2018, 168: 487-504.

449 Nakashima, M., Inoue, K., and Tada, M. (1998). Classification of damage to steel buildings
 450 observed in the 1995 Hyogoken-Nanbu earthquake. *Eng. Struct.*, 20(4–6), 271–281.

451 Ricles, J., Sause, R., Peng, S., and Lu, L. (2002). Experimental Evaluation of Earthquake
 452 Resistant Posttensioned Steel Connections. *J. Struct. Eng.*, 128(7).

453 Park, J., Kang, S., Yang, S. (2005). Experimental studies of wide flange beam to square
 454 concrete-filled tube column joints with stiffening plates around the column. *J. Struct. Eng.*,
 455 131(12).

456 Qin, Y., Chen, Z., Yang, Q., Shang, K. (2014). Experimental seismic behavior of
 457 through-diaphragm connections to concrete-filled rectangular steel tubular columns.
 458 *Journal of Constructional Steel Research*, 93:32-43.

459 Qin, Y., Chen, Z., Wang, X. (2014). Experimental investigation of new internal-diaphragm
 460 connections to CFT columns under cyclic loading. *Journal of Constructional Steel*
 461 *Research*, 98: 35-44.

462 Rong, B., Liu, S., Yan, J., and Zhang R. (2018). Shear behaviour of panel zone in
 463 through-diaphragm connections to steel tubular columns. *Thin-Walled Structures*, 122:
 464 286-299.

465 Rong, B., Yang, Z., Zhang, R., and Feng, C. (2019). Postbuckling shear capacity of external
 466 diaphragm connections of CFST structures. *J. Struct. Eng.*, 145(5).

467 Shin, K., Kim, Y., Oh, Y., Moon, T. (2004). Behavior of welded CFT column to H-beam
 468 connections with external stiffeners. *Eng. Struct.*, 26(13): 1877-1887.

469 Suita K, Yamada S, Tada M, Kasai K, Matsuoka Y, Shimada Y. (2008). Collapse experiment
 470 on 4-story steel moment frame, part 2 detail of collapse behavior. *Proceedings of the 14th*
 471 *World Conference on Earthquake Engineering*, Beijing.

472 Takewaki, I., Murakami, S., Fujita, K., Yoshitomi, S., Tsuji M. (2011). The 2011 off the Pacific
 473 coast of Tohoku earthquake and response of high-rise buildings under long-period ground
 474 motions. *Soil Dynamics and Earthquake Engineering*, 31(11), 1511-1528.

475 Toshiyuki Fukumoto, T. and Morita, K. (2005). Elastoplastic behavior of panel zone in steel
 476 beam-to-concrete filled steel tube column moment connections. *J. Struct. Eng.*, 131(12).

477 Uang, C., Yu, Q., Noel, S. (2000) Cyclic testing of steel moment connections rehabilitated with
 478 RBS or welded haunch, *J. Struct. Eng.*, 126(1): 57-68.

479 Vulcu, C., Stratan, A., Ciutina, A., and Dubina, D. (2017). Beam-to-CFT high-strength joints
 480 with external diaphragm. I: Design and experimental validation. *J. Struct. Eng.*, 143(5).

481 Wang, W., Chan, T., Shao, H., and Chen, Y. (2015). Cyclic behavior of connections equipped
 482 with NiTi shape memory alloy and steel tendons between H-shaped beam to CHS column.
 483 *Eng. Struct.*, 88: 37-50.

484 Yoshizato, J., Nakamura, Y., Kuromatsu, J., Matsuo, A. and Owa, Y. Experimental Study on
 485 SHS Column to H-Beam Connections with exterior Plate Diaphragms, *Summaries of*
 486 *technical papers of Annual Meeting*, AIJ, Structures II, 1994, 1417-1418. (in Japanese)

487 Yu, Y., Lan, L., Chen, Z., and Huang, J. (2019). Mechanical and seismic behaviors of
 488 bottom-flange-bolted upper-flange-welded through-diaphragm connections. *Journal of*
 489 *Constructional Steel Research*, 156: 86-95.

Figures:

Fig. 1 Plan and elevation views of the specimen (No.1)

(a) Type A

(b) Type B

Fig. 2 Dimension of the external stiffener rings

Fig. 3 Overall view of the test setup

(a) Drift controlled deformation

(b) Time history of cycle

Fig. 4 Mechanism of bidirectional cyclic loading

(a) Elevation view in west direction

(a) Elevation view in south direction

Fig. 5 Measurement systems for various displacements

Fig. 6 Beam moment-rotation angle curves of specimens: (a) Specimen No. 1 (beam 300×120×6×12, bi-direction, weld 10 mm); (b) Specimen No. 2 (beam 300×120×6×12, uni-direction, weld 10 mm); (c) Specimen No. 3 (beam 200×120×6×12, bi-direction, weld 10 mm); (d) Specimen No. 4 (beam 450×120×6×12, bi-direction, weld 10 mm); and (e) Specimen No. 5 (beam 300×120×6×12, bi-direction, weld 6 mm)

(a) Inward buckling of column in No.1

(b) Crack at column base of specimen No.1

(c) Overview of deformed specimen No. 2

(d) Inward buckling of column in No. 2

(e) Local buckling of specimen No. 3

(f) Inward buckling of column in No. 3

(g) Inward buckling of column in No.4

(h) Fracture of the ring stiffener ring in No. 4

(i) Inward buckling of column in No. 5

(j) Fracture of weld fillet in No. 5

Fig. 7 Failure modes of specimens

(a) Specimen No. 1

(b) Specimen No. 3

(c) Specimen No. 4

(d) Specimen No. 5

Fig. 8 Skeleton curves of double direction loading specimens

Fig. 9 Effects of beam depth on elastic stiffness and flexural strength capacity

Fig. 10 Effect of loading mode on flexural strength

Fig. 11 Influence of the fillet weld leg size on bearing capacity

Fig. 12 Definition of yielding and full-plastic points

(a) No.1-west beam

(b) No.1-south beam

(c) No.2

(d) No.3-west beam

(e) No.3-south beam

(f) No.4-west beam

(g) No.4-south beam

Fig. 13 K_j - θ curves at the beam ends of various test specimens

Fig. 14 Calculation for equivalent viscous damping coefficient

Fig. 15 Progressive energy dissipations with the rotation angle

Fig. 16 ζ_{eq} - θ curves of specimens

Fig. 17 Location of strain gauges on the outer stiffened ring

Fig. 18 Strain distribution of external stiffener rings

Fig. 19 The shear deformation of panel zones

(a) No. 1

(b) No. 2

(c) No. 3

(a) No. 4

(b) No. 5

Fig. 20 The shear deformation angel of panel zones

(a) No.1

(b) No.3

(c) No.4

(d) No.5

Fig. 21 Sear strain values in panel zone

(b) External diaphragm under bidirectional load

(b) Plastic mechanism of panel zone

Fig. 22 Failure mechanisms of external diaphragm and panel zone in joint