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(Citation)

Japan Architectural Review, 9(1):e70104

(Issue Date)

2026-01

(Resource Type)

journal article

(Version)

Version of Record

(Rights)

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ORIGINAL PAPER **OPEN ACCESS**

Effect of Concrete Strength on Joint Performance of Orthogonal Split Beam Joints in Continuous Beam-Type Circular CFST Beam-To-Column Connections

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ABSTRACT

In a continuous beam-type connection, a form of concrete-filled steel tubular (CFST) beam-to-column connection, fixing the beam ends becomes problematic because a beam orthogonal to the continuous beam cannot pass through the column tube. This study discusses methods for connecting beams orthogonal to the continuous beam in continuous beam-type CFST beam-to-column connections and investigates joint performance under relatively low filled concrete strength. A sub-assembly test is conducted using high-strength concrete to investigate connection behavior and the effect of concrete strength on performance. Additionally, finite element analysis is performed to examine the stress transfer mechanism within and around the connection panel. Based on the experimental and analytical results, the stress transfer mechanism within and around the connection panel is discussed with respect to differences in connection details. Furthermore, a schematic model illustrating the assumed stress transfer mechanism associated with bearing resistance is presented.

1 | Introduction

Concrete-filled steel tubular (CFST) structures have been widely used as columns in building structures because of their excellent structural performance. In CFST structures, three typical types of beam-to-column connections are used: through-diaphragm, outer-diaphragm, and inner-diaphragm. Design methods for these connection types have been established [1]. Among the three connection types, the through-diaphragm type is the most commonly adopted. However, the design of the diaphragm plate is relatively complex due to the presence of perforation for concrete infill.

A continuous beam-type connection, in which a steel beam passes through a steel tube column, has been proposed [2]. Furthermore, a stress-transfer mechanism and corresponding design formulas have been established [3]. The authors have investigated the performance of continuous beam-type CFST beam-to-column connections with a square column

cross-section [4, 5]. However, the performance of connections in the direction orthogonal to a continuous beam has not been sufficiently investigated. A connection method for the orthogonal direction has been proposed, and the performance of split-beam connections orthogonal to the continuous beam in circular CFST beam-to-column connections has been experimentally and analytically investigated for cases involving relatively low concrete compressive strength [6, 7]. However, highly workable concrete is often used in CFST structures to improve filling performance; consequently, the infilled concrete tends to exhibit high compressive strength.

In this study, sub-assembly tests were conducted on split-beam connections orthogonal to a continuous beam in circular CFST beam-to-column connections under relatively high concrete compressive-strength conditions. The fundamental joint performance and the effect of concrete strength on the joint behavior of split-beam connections orthogonal to a continuous

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beam were examined through comparison with the experimental results reported in a previous study [7]. Additionally, finite element (FE) analyses were performed to investigate the stress-transfer mechanism within and around the connection panel.

2 | Sub-Assemblage Test for Beam-To-Column Connections

2.1 | Specimens

Figure 1 illustrates the geometry and dimensions of the specimens, while Figure 2 presents the joint details. Table 1 lists the specimens, and Table 2 summarizes the material properties

of the steel. Four specimens were tested to simulate beam-to-column connections between circular CFST columns and steel beams. Consistent with previous studies [6, 7], an I-section beam was inserted through a beam-shaped slot cut into a steel tube, and a fillet weld was applied from the exterior surface of the tube. In Figure S1, the transverse continuous beam is indicated in blue, whereas the split-beam running orthogonally to it is indicated in red. The circular column tube had a cross-section of 318.5×6 (STK400), and the I-section steel beam was $H-300 \times 150 \times 6.5 \times 9$ (SS400). Three types of transverse continuous beams were used: $H-300 \times 150 \times 6.5 \times 9$, $H-400 \times 200 \times 8 \times 13$, and $H-350 \times 175 \times 7 \times 11$, with the flange widths of the latter two I-section beams reduced to 150 mm. All specimens were designed based on an assumed beam-yielding failure mechanism.

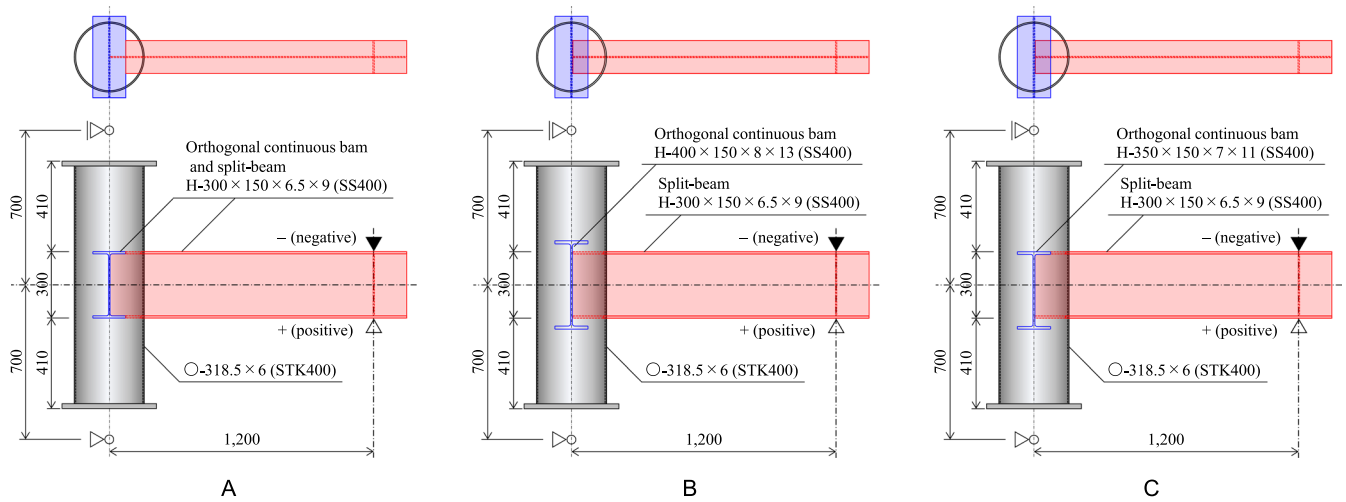


FIGURE 1 | Specimen dimensions: (A) Specimen O-60-300, (B) Specimen O-60-400, and (C) Specimen O-60-350A and O-21-350A.

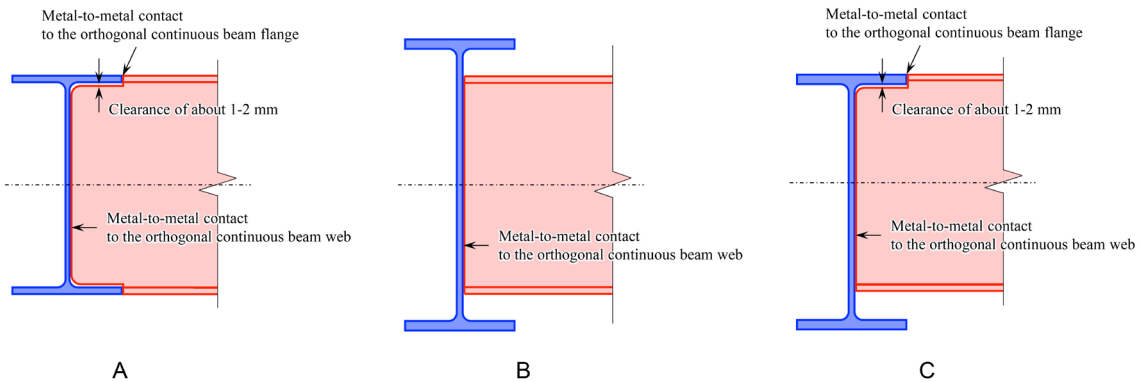


FIGURE 2 | Joint details: (A) Specimen O-60-300, (B) Specimen O-60-400, and (C) Specimen O-60-350A and O-21-350A.

TABLE 1 | List of specimens.

Specimens	Column	Split-beam	Orthogonal continuous beam	Concrete compressive strength (MPa)
O-60-300	318.5×6	H-300×150×6.5×9	H-300×150×6.5×9	72.8
O-60-400	318.5×6	H-300×150×6.5×9	H-400×150×8×13	72.8
O-60-350A	318.5×6	H-300×150×6.5×9	H-350×150×7×11	71.6
O-21-350A	318.5×6	H-300×150×6.5×9	H-350×150×7×11	27.8

TABLE 2 | Mechanical properties of steel.

		Elastic modulus (GPa)	Yield strength (MPa)	Ultimate strength (MPa)	Elongation (%)
H-300×150×6.5×9	Flange	187	333	450	27.5
H-300×150×6.5×9	Web	186	381	475	24.3
H-400×150×8×13	Flange	182	305	441	31.1
H-400×150×8×13	Web	186	382	484	23.3
H-350×150×7×11	Flange	184	311	452	29.4
H-350×150×7×11	Web	180	355	477	23.6
318.5×6	Column tube	193	377	468	33.7

For specimens O-60-300, O-60-350A, and O-21-350A, a portion of the split-beam flange overlapping the continuous beam was removed, and the web of the inserted split-beam was brought into direct metal-to-metal contact with the web of the transverse continuous beam. A clearance of 1–2 mm was provided between the flange of the transverse beam and the web of the inserted split-beam. The experimental parameters included the depth difference between the continuous and split beams, and the design compressive strength of the concrete infill. The concrete compressive strengths were 60 and 21 MPa. Concrete was cast using a conventional pouring method.

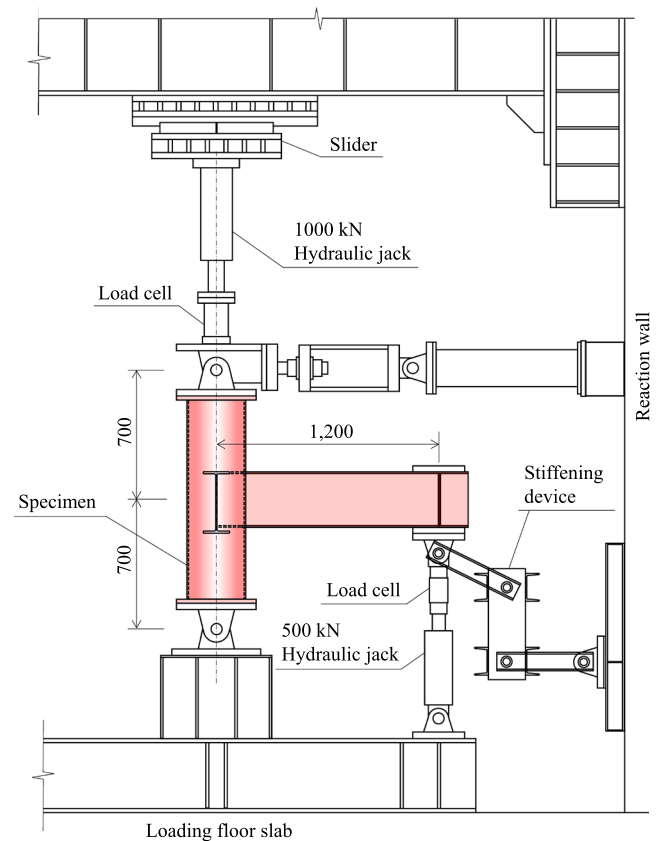
2.2 | Loading and Measurement Methods

Figure 3 illustrates the loading apparatus used in this study. The top and bottom ends of the column were supported by a roller and a hinge, respectively. An axial force corresponding to 10% of the column squash load was applied to the column, and a cyclic shear force was applied using a hydraulic jack positioned at the beam end. Stiffening devices were installed at the beam end to restrain out-of-plane displacement. Loading was controlled by rotational angle with target values of 0.005, 0.01, 0.02, 0.03, and 0.04 rad. Two cycles were applied for each target rotation angle of $R=0.005$, 0.01, 0.02, and 0.03 rad, whereas only one cycle was applied for $R=0.04$ rad. R was calculated based on the vertical displacement at the beam end and the distance between the loading point and the column center (1200 mm). The shear force direction was defined as positive when a sagging moment occurred at the end of the split beam. The vertical displacement at the beam end was measured on the upper surface of the split I-section beam, using the column as the reference point. Strain gauges were used to measure strain at various locations.

3 | Experimental Results

3.1 | Relationship Between the Shear Force and Rotation Angle

Figure 4 presents the experimentally obtained shear force–rotation angle relationship. In the figure, the solid red lines represent the experimentally obtained response. The black triangles and squares indicate the initiation of yielding in the beam flange

**FIGURE 3** | Loading apparatus.

on the tension and compression sides, respectively, at a cross section located 75 mm from the outermost surface of the steel tube column. The black circles denote peak-strength points, and the arrows indicate the onset of local buckling in the flanges of the I-section beam. The green and blue lines correspond to shear forces calculated from the yield and ultimate flexural strengths of the I-section beams at the surface of the steel tube column, respectively.

Specimen O-60-400, with a transverse beam depth of 400 mm, exhibited stable hysteretic behavior with high energy dissipation capacity. Specimens O-60-300 and O-60-350A exhibited good energy-dissipating hysteretic behavior at small rotation

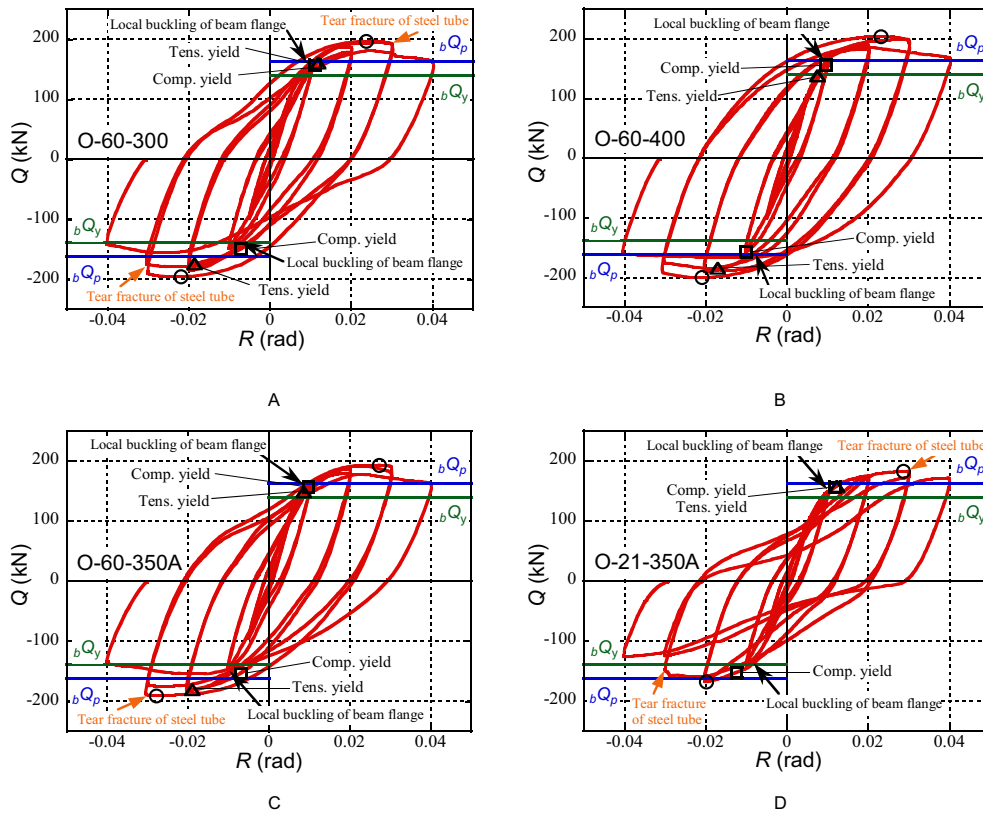


FIGURE 4 | Relationship between shear force and rotation angle: (A) Specimen O-60-300, (B) Specimen O-60-400, (C) Specimen O-60-350A, and (D) Specimen O-21-350A.

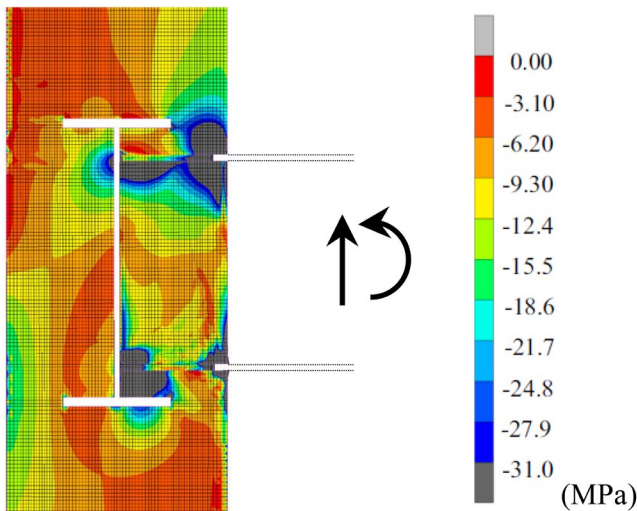


FIGURE 5 | Minimum principal stress distribution of infilled concrete (Specimen O-21-400, $R=0.02$ rad) [7].

angles. However, beyond a rotation angle of approximately $R=0.03$ rad, the hysteresis loops exhibited a slightly pinched geometry during the reloading phase after unloading. Specimen O-21-350A, filled with Fc21 concrete, exhibited pinched hysteretic behavior with lower energy absorption capacity. The larger the flange area in the embedded beam region, the more stable the hysteresis loops, indicating that bearing resistance provided by the surrounding concrete near the flange within the connection panel significantly governs behavior in large deformation

ranges. FE analysis results reported in a previous study [7] revealed the bearing resistance mechanism around the flange and tube wall on the compression side of the connection panel (Figure 5). When the concrete strength is high and/or the beam embedment length is large, rotational restraint due to bearing resistance increases, and deformation caused by plasticization at the beam end becomes more dominant relative to overall frame deformation. Conversely, when concrete strength is low and/or beam embedment length is short, rotational restraint decreases, and bearing resistance governed by concrete behavior controls the overall deformation, resulting in pinched hysteretic behavior.

Figure 6 presents a comparison of the shear force–rotation angle relationship, highlighting the influence of concrete strength. The ordinate represents the beam shear force normalized by the frame strength calculated using the full plastic flexural strength of the beam. The compressive strength of the infilled concrete used in the previous study [7] was approximately 30 MPa.

When comparing specimens with identical connection details, no influence of concrete strength was observed on shear strength or hysteretic energy dissipation within the small deformation range up to a drift ratio of $R=0.01$ rad. However, in the large deformation range (beyond $R=0.02$ rad), the specimens filled with Fc21 concrete and having cut-off flange ends within the connection panel (specimens O-21-300 and O-21-350A) showed a pronounced reduction in both strength and hysteretic energy dissipation. Reduced resistance during

reloading (second and fourth quadrants) led to an increase in the proportion of rigid-body rotation relative to total deformation, thereby inhibiting sufficient plasticization at the end of the steel beam. However, compared with the O-21 series, the O-60 series exhibited only minor pinching in the hysteresis loops. In particular, the hysteresis loop of O-60-400 exhibited high energy dissipation with no noticeable pinching. In the O-60 series, rotational restraint provided by concrete bearing resistance was significant. For O-60-400, a concrete compressive strut formed between the transverse continuous beam and the main split-beam enhanced beam fixity and promoted plastic deformation at the beam ends.

3.2 | Comparison of Strength

Table 3 presents a comparison between the experimental and calculated results. The calculated yield strength and ultimate

strength correspond to the flexural yield strength and full plastic flexural strength of the steel beam, respectively, obtained by dividing these values by the distance from the loading point to the outermost face of the steel tube column. The yield strength was predicted within 15% accuracy for all specimens. In the O-60 series, the experimental peak strengths exceeded the calculated values by 20%–30% for all specimens. Conversely, in the O-21 series, except for O-21-400 with a larger depth of the transverse continuous beam, the experimental values exceeded the calculated values by 10%–15%.

The strengths of O-21-300 and the negative-direction response of O-21-350A were approximately 10% lower than those of the other specimens. In these specimens, the beam flanges at the embedded ends were removed such that the concrete bearing resistance acted only in the shallow embedded region. Specimens with cut-off flanges exhibited less restraint provided by the steel beam, making plastic deformation at the beam ends more

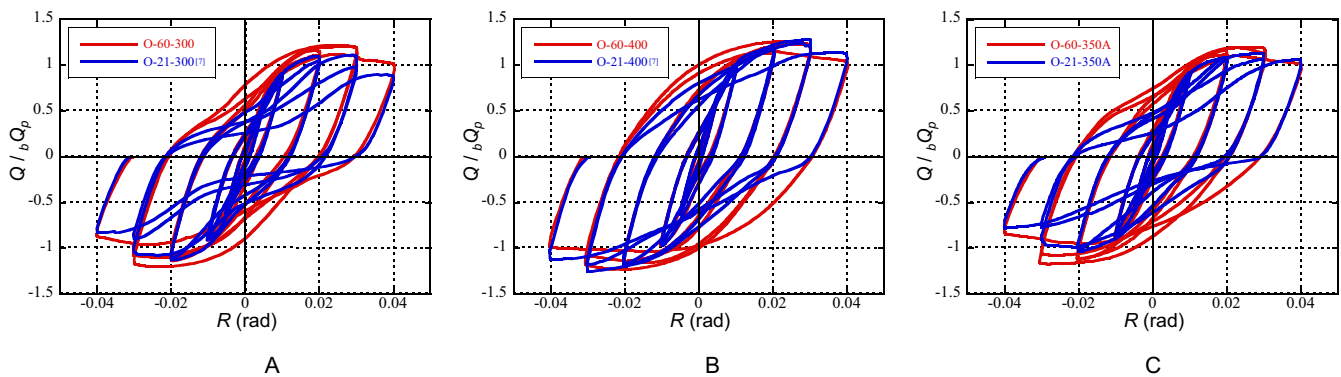


FIGURE 6 | Comparison of the relationship between the shear force and rotation angle [7]: (A) O-300 series, (B) O-400 series, and (C) O-350A series.

TABLE 3 | Comparison between experimental and calculated results.

Specimens	eQ_y (kN)	bM_y (kNm)	bQ_y (kN)	eQ_y/bQ_y	eQ_u (kN)	bM_p (kNm)	bQ_p (kN)	eQ_u/bQ_p	cM_p (kNm)	cQ_u (kN)
O-60-300	161	144	139	1.16	197	163	156	1.26	352	677
	−147	144	139	−1.06	−195	163	156	−1.25	352	677
O-60-400	152	145	139	1.10	203	163	156	1.30	352	677
	−157	145	139	−1.13	−200	163	156	−1.28	352	677
O-60-350A	149	145	140	1.06	193	164	158	1.22	352	677
	−155	145	140	−1.10	−191	164	158	−1.21	352	677
O-21-300 [7]	142	149	143	0.99	180	169	162	1.11	304	645
	−143	149	143	−1.00	−184	169	162	−1.14	304	645
O-21-400 [7]	147	149	143	1.03	207	169	162	1.28	306	649
	−157	149	143	−1.10	−204	169	162	−1.26	306	649
O-21-350A	158	144	138	1.14	183	162	156	1.17	295	567
	−155	144	138	−1.13	−168	162	156	−1.08	295	567

Abbreviations: bM_y , bM_p , yield flexural strength and full plastic flexural strength of the steel beam, respectively; bQ_y , bQ_p , shear force corresponding to the yield flexural strength and the full plastic flexural strength of the steel beam, respectively; cM_p , ultimate flexural strength of the CFST column; cQ_u , shear force corresponding to the ultimate flexural strength of the CFST column; eQ_y , eQ_u , experimental yield strength and experimental peak strength, respectively.

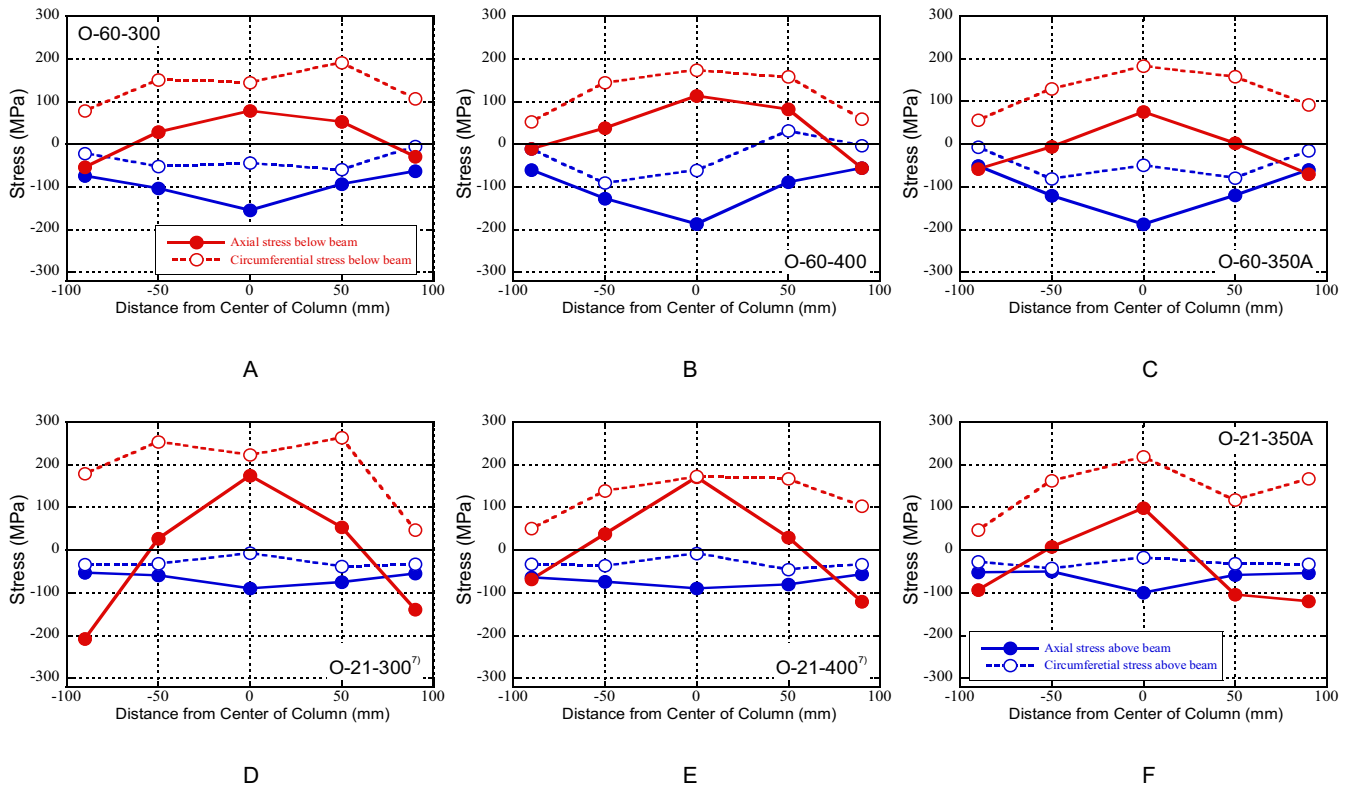


FIGURE 7 | Stress distribution on the steel tube column surface in the axial and circumferential directions during positive loading (first cycle at $R=0.005$ rad): (A) Specimen O-60-300, (B) Specimen O-60-400, (C) Specimen O-60-350A, (D) Specimen O-21-300 7, (E) Specimen O-21-400 7, and (F) Specimen O-21-350A.

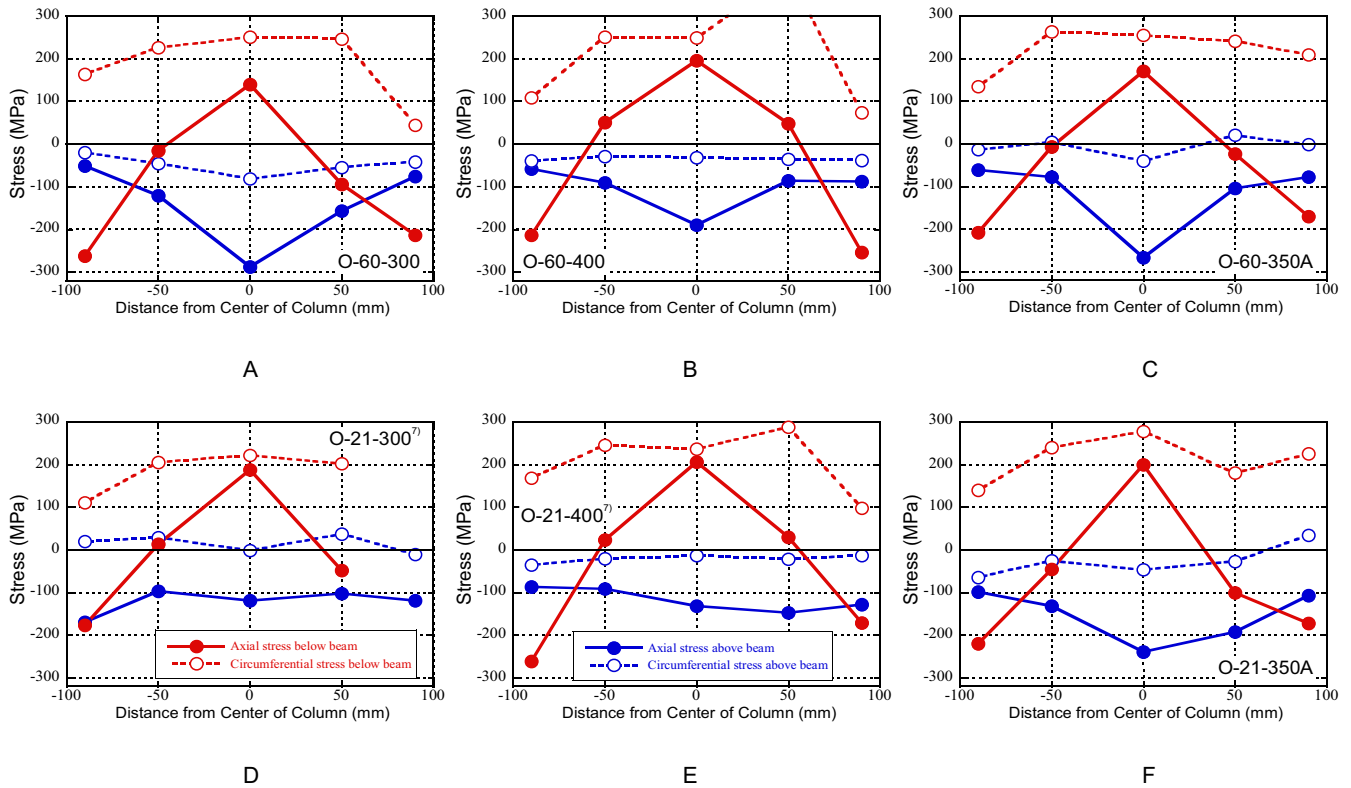


FIGURE 8 | Stress distribution on the steel tube column surface in the axial and circumferential directions during negative loading (first cycle at $R=-0.005$ rad): (A) Specimen O-60-300, (B) Specimen O-60-400, (C) Specimen O-60-350A, (D) Specimen O-21-300 7, (E) Specimen O-21-400 7, and (F) Specimen O-21-350A.

difficult to develop. However, at higher concrete strength, a certain degree of restraint was expected even with a small bearing area, allowing plastic deformation at the beam end. Therefore, even in O-60-300 and O-60-350A, where the beam flanges at the embedded ends were cut off, the experimental values exceeded the calculated values by 20%–25%.

3.3 | Stress Distribution of the Steel Tube Above and Below the Beam

Figures 7 and 8 present the stress distributions on the steel tube surface above and below the split I-section beam during the first cycle of positive and negative loading at $R = \pm 0.005$ rad. Assuming a plane stress state, axial and circumferential stresses were calculated from strain values measured using biaxial strain gauges attached to the steel tube surface 25 mm above and below the split-beam flanges (Figure 9). The red and blue lines represent data from the bending-tension and bending-compression side surfaces below and above the beam, respectively.

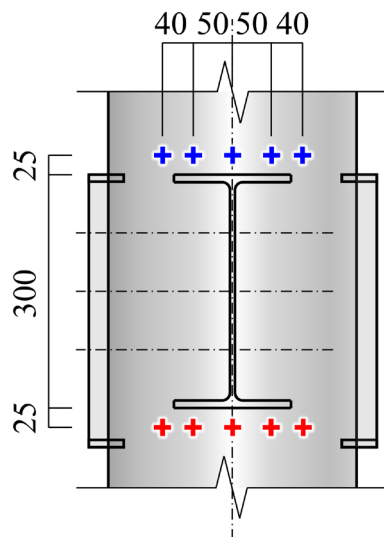


FIGURE 9 | Strain gauge positions on the column tube surface above and below the I-section beam.

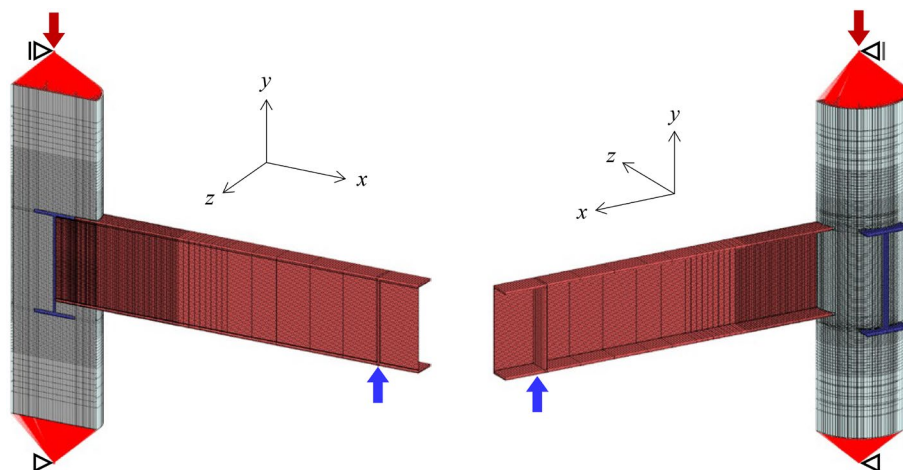


FIGURE 10 | Example of finite element meshing (Specimen O-60-350A).

Circumferential stress was low in all specimens on the bending-tension side; however, it became dominant on the bending-compression side because the compressive force from the steel beam flange transferred directly to the infilled concrete through the steel tube wall. Conversely, the tensile force from the beam flange was primarily transferred through circumferential stress action in the steel tube column. As the concrete strength increased, axial stress decreased slightly, whereas compressive stress at the center of the I-section beam on the bending-compression side (the upper and lower sides under positive and negative loading, respectively) increased. This compressive stress corresponds to the reaction force generated when the embedded split-beam resists bearing pressure; higher concrete strength enhances bearing resistance, resulting in larger reaction forces.

4 | Stress-Transfer Mechanism Around the Connection Panel

4.1 | Analytical Modeling

A nonlinear FE analysis was performed using MSC.MARC [8]. Figure 10 illustrates a representative FE mesh of the specimen. The steel tube, I-section beam, and infilled concrete were modeled using eight-node hexahedral or six-node wedge isoparametric solid elements. A friction coefficient of 0.3 was assigned to the steel-concrete interface, which is commonly used in analyses of steel-concrete composite structures [7, 9–11]. The fillet weld between the steel tube column and the steel beam was modeled using the GLUE option. A perfectly elastic-plastic stress-strain relationship was assumed for the steel material model. The constitutive law for the plastic region was based on the von Mises yield criterion. Figure 11 presents the uniaxial stress-strain curve for the infilled concrete, plotted using data obtained from the cylinder compression test up to the compressive strength. A modified Ahmad model was employed to define the post-peak stress-strain relationship [12]. Young's modulus (E) was derived from the unloading curve of the stress-strain relationship obtained from the cylinder test up to a stress level corresponding to one-third of the compressive strength (f_c). The tension-softening modulus (E_{soft}) was assumed to be 20% of E . The tensile strength (f_t) was determined from the splitting

tensile strength of the cylinder. The constitutive law for the plastic region was based on the Mohr-Coulomb's failure criterion with an internal friction angle (ϕ) of 30° and an α value of 0.16. The smeared crack model was used to analyze concrete fracture, with a shear retention factor of 0.5 adopted to account for residual shear transfer across cracks.

4.2 | Analytical Results

The shear force–rotational angle relationships obtained from the FE analyses are compared with the experimental envelope curves, as shown in Figure 12. The analytical results were in relatively good agreement with the experimental values, particularly up to $R=0.015$ rad.

4.2.1 | Stress Distribution and Deformation of the Steel Part

Figure 13 shows the von Mises stress distributions in the steel tube column and beam at $R=0.02$ rad. At $R=0.005$ rad, yielding initiated at the end of the split-beam flange. By $R=0.02$ rad, the plastic region at the split-beam end had expanded. The O-60 series with high-strength concrete exhibited a slightly

larger plasticized region than the O-21 series. Additionally, the 400 and 350A series under positive loading, which exhibited a larger flange area capable of providing bearing resistance within the connection panel against rigid-body rotation of the split-beam, demonstrated a larger plastic region than the 300 and 350A series under negative loading. In all specimens, the tube wall around the flange on the bending-tension side deformed and experienced increased stress due to flange pull-out. Figure 14 shows the local deformation around the tension-side flange. Because the tube wall provides minimal resistance to out-of-plane deformation, the flange on the bending-tension side was significantly pulled out, and the circumferential in-plane action of the tube wall resisted the resulting tensile force.

4.2.2 | Stress Distribution in Filled Concrete and Resisting Mechanism in the Connection Panel

Figure 15 shows the stress distributions in the infilled concrete at $R=0.02$ rad on the section cut along the web plane of the split-beam. From the minimum principal stress distributions, a compression strut was formed between the split-beam flange and the continuous beam flange in the perpendicular direction in O-60-400 and O-60-350A under positive loading. Similar observations were made in O-21-350A. Furthermore, the specimens in which the compression strut formed exhibited a greater degree of plasticization at the split-beam end. Vertical bearing resistance developed above and below the flange embedded in the split-beam connection panel, contributing to rotational restraint. On the bending-compression side, large compressive stresses were transmitted from the split-beam flange to the infilled concrete through the tube wall. Horizontal bearing resistance contributed significantly to the stress transfer mechanism.

From the resistance mechanism illustrated in Figure 15, the resisting distribution shown in Figure 16 can be inferred. The blue regions represent vertical bearing resistance transmitted from the split-beam flange to the infilled concrete, while the red regions represent horizontal bearing resistance transmitting axial force from the split-beam flange through the tube wall. According to the bearing stress distribution shown in Figure 16, flexural resistance due to bearing within

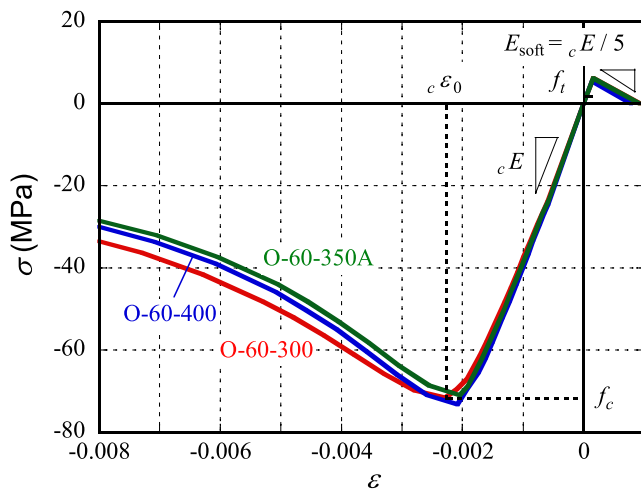


FIGURE 11 | Stress–strain curve of infilled concrete.

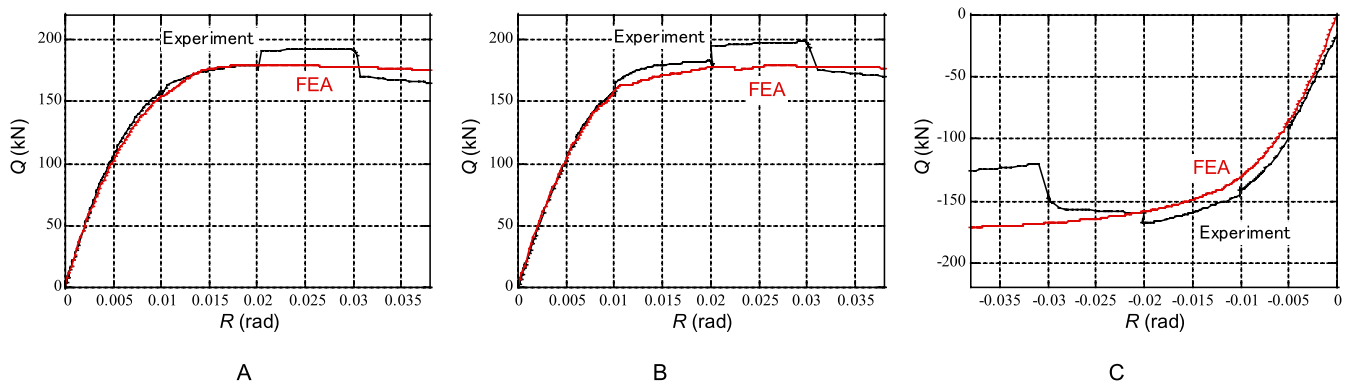


FIGURE 12 | Comparison between experimental and analytical behaviors: (A) Specimen O-60-350A (positive loading), (B) O-60-400 (positive loading), and (C) Specimen O-21-350A (negative loading).

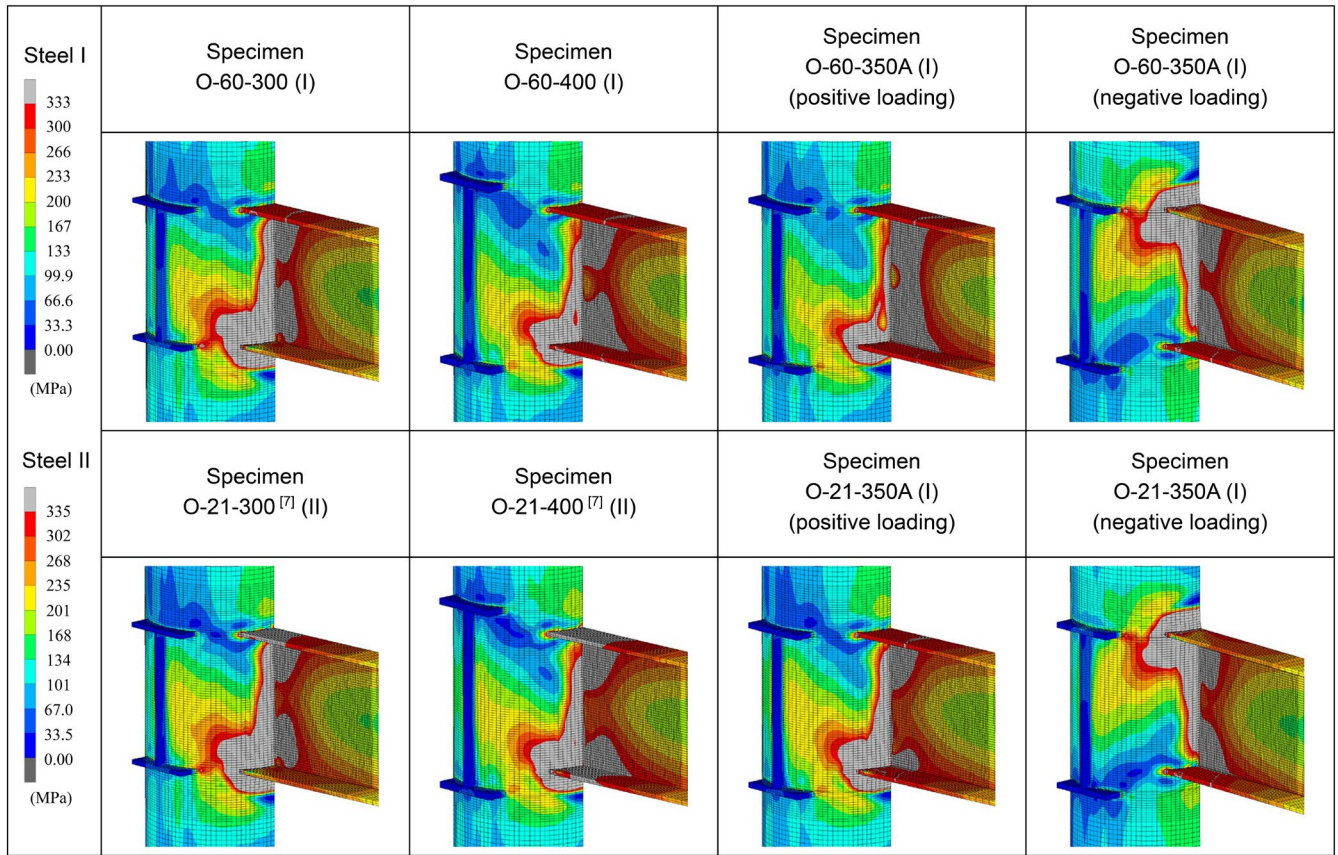


FIGURE 13 | von Mises stress distribution on the steel tube and beam ($R=0.02$ rad).

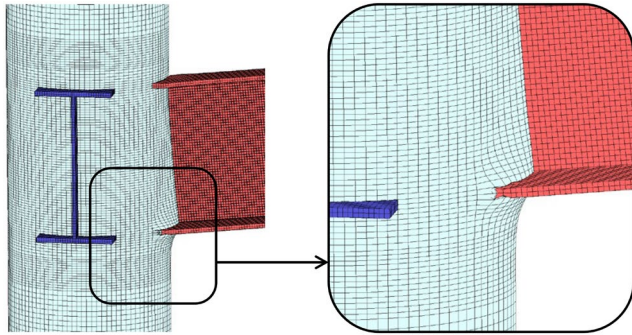


FIGURE 14 | Local deformation around the tension-side flange at $R=0.02$ rad (Specimen O-60-300, deformation magnified three times).

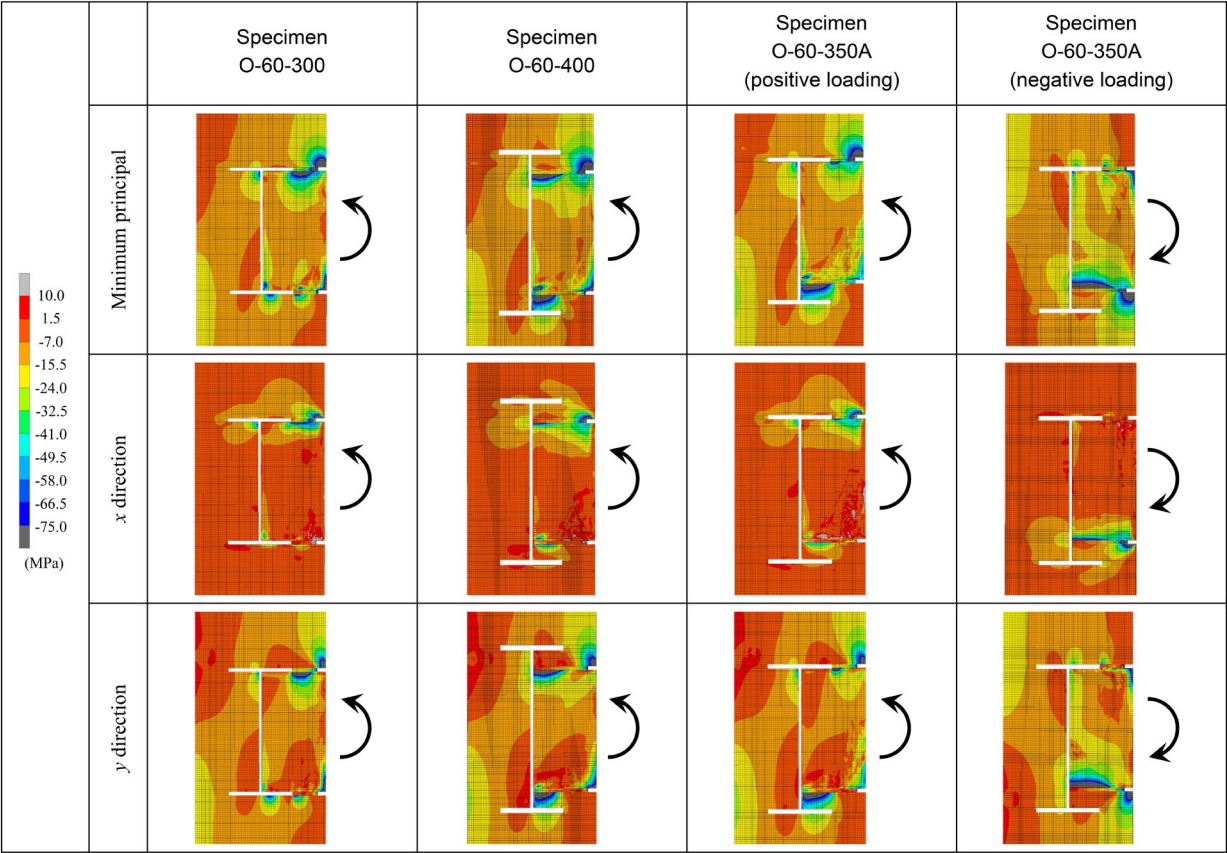
the connection panel can be evaluated as the sum of (i) flexural resistance resulting from bearing transferred from the bending-compression side flange to the concrete through the steel tube wall and (ii) flexural resistance due to bearing between the beam flange and the concrete within the connection panel. Accordingly, the flexural resistance can be expressed as Equation (1):

$$M_b = {}^{nw}M_b + {}^{uf}M_b + {}^{lf}M_b = \left(\frac{B^2 \cdot H}{4} + \frac{B \cdot x_U^2}{6} + \frac{B \cdot x_L^2}{6} \right) f_c \quad (1)$$

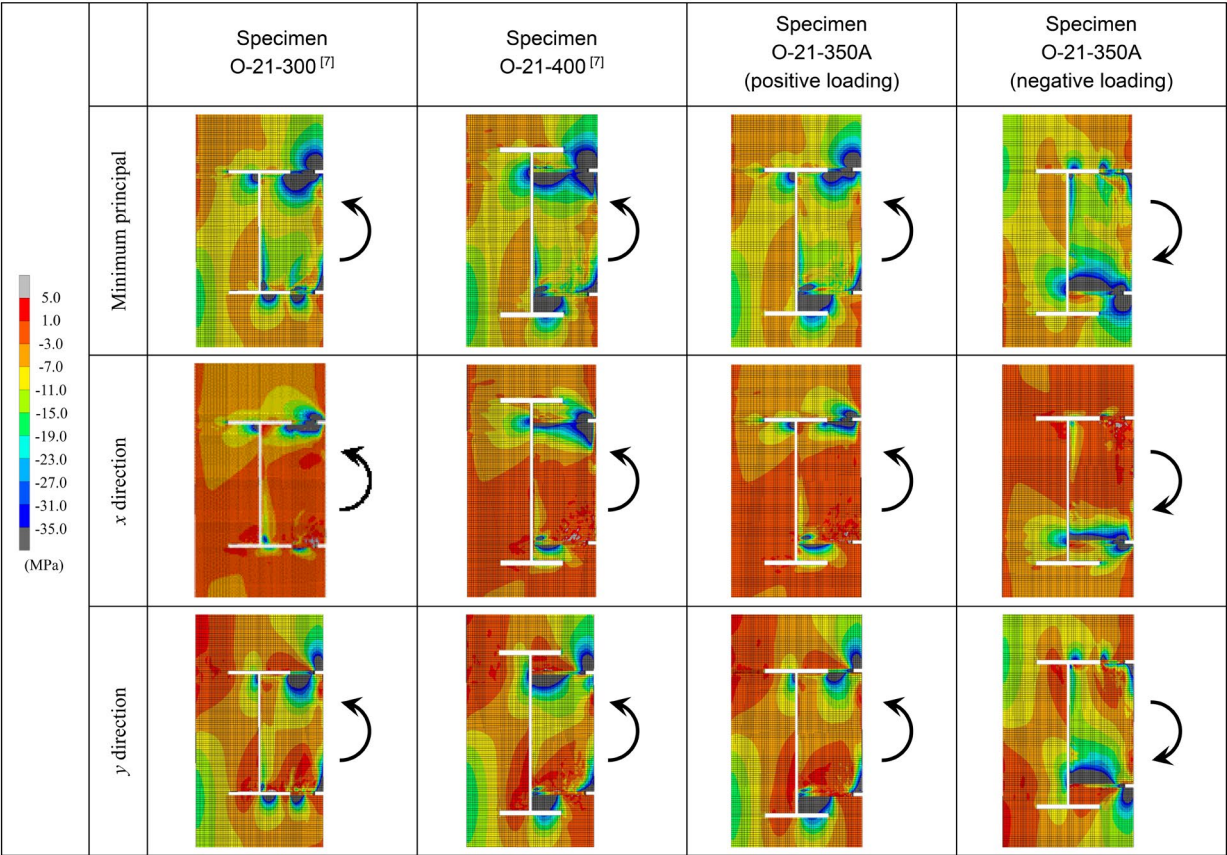
where B represents the beam width, and x_U and x_L denote the embedded lengths of the upper and lower flanges, respectively. Bearing stress was equated to the compressive strength

of concrete (f_c), and the bearing width was taken as the beam width. The height of the bearing stress block on the bending-compression side, transferred from the compression flange through the steel tube wall, was assumed to be half the beam width. For simplicity, the curvature of the steel tube wall was neglected in the calculation. The difference between the calculated values with and without considering the curvature was approximately 5%. Furthermore, when the bending-tension side corresponded to the side where a portion of the flange was cut, bearing resistance was neglected based on FE analysis results, indicating low stress in that region.

Table 4 summarizes the calculated flexural resistance around the connection panel due to bearing action. As shown in the table, O-60-400 and O-60-350A, which exhibited superior energy absorption capacity, developed sufficient flexural resistance due to bearing action to reach full beam moment capacity. Additionally, O-60-300 and O-21-400, which demonstrated relatively good energy absorption performance, exhibited flexural resistance corresponding to approximately 80% of the beam moment capacity. It should be noted that the rotational restraint of the beam is not governed solely by the bearing resistance of concrete; the steel tube column and the continuous transverse beam also contribute to the restraint mechanism. Nevertheless, only bearing resistance was considered in the present evaluation. However, bearing stress distribution was expected to vary depending on the beam embedment length, beam and column dimensions, and concrete strength. Therefore, the evaluation is considered valid only for the specimens examined in this study.



A



B

FIGURE 15 | Minimum principal stress distribution of infilled concrete ($R=0.02$ rad).

4.2.3 | Rigid-Body Rotation due to Pull-Out of the Split Beam

Figure 17 presents the relationship between rigid-body rotation of the split-beam and total frame deformation. Rigid-body rotation was derived from the FE analysis results by dividing the relative displacement between the upper and lower flanges of the split-beam, measured at the outermost surface of the steel tube column, by the beam depth (Figure 18). The proportion of rigid-body rotation in total frame deformation was

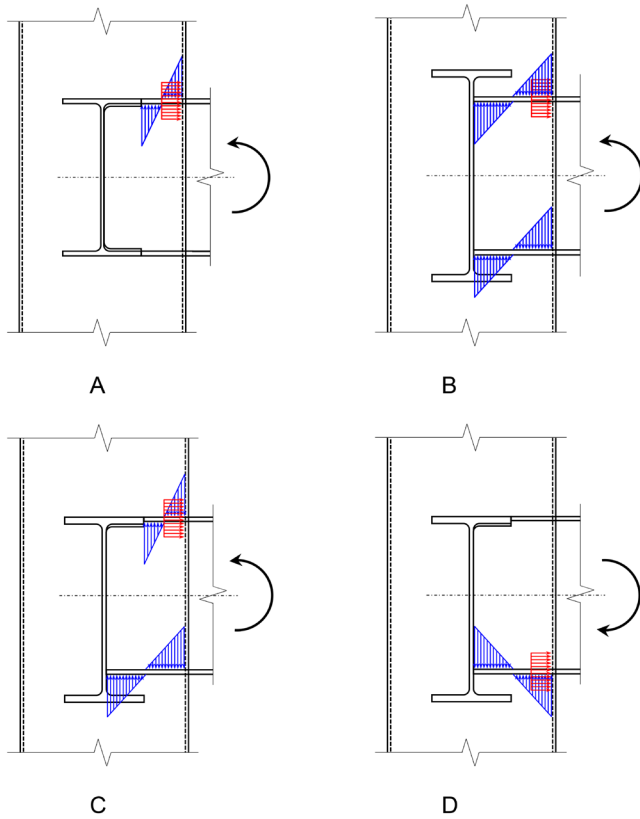


FIGURE 16 | Schematic of the resisting mechanism: (A) O-300 series, (B) O-400 series, (C) O-350A series (positive loading), and (D) O-350A series (negative loading).

relatively large for the 300 series, and the negative loading of the 350A series, in which the flange was cut within the connection panel, resulting in a reduced bearing area. Conversely, for the 400 series and the positive loading case of the 350A series, where the flange provided a larger bearing area within the connection panel, the proportion of rigid-body rotation decreased with increasing total frame deformation. In particular, for high concrete strength, the increase in rigid-body rotation was limited beyond $R = 0.015$ rad. This finding suggests that subsequent deformation is mainly attributable to plastic rotation at the beam end, rather than additional rigid-body rotation. The observed tendency is consistent with experimental observations.

5 | Conclusions

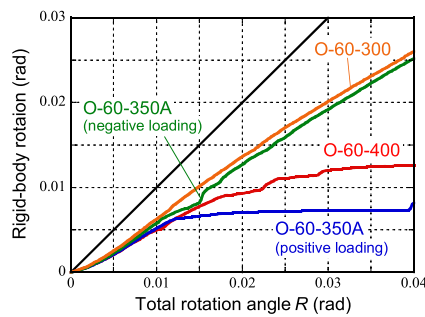
Sub-assembly tests and FE analyses were conducted to investigate the fundamental performance of split-beam connections orthogonal to a continuous beam in circular CFST beam-to-column connections using relatively high-strength concrete. The following conclusions are drawn from the experimental and analytical results.

1. All specimens exhibited maximum strengths exceeding the calculated values and stable hysteretic behavior without sudden strength degradation, even after cracks formed at the weld ends around the steel beam flanges and steel tube columns. Hysteretic behavior was strongly influenced by concrete strength. Specimens with high-strength concrete exhibited more stable hysteresis loops and greater energy dissipation capacity than specimens with $F_c = 21$ MPa concrete, owing to the enhanced rotational restraint of the split-beam attributable to bearing resistance.
2. FE analyses revealed the stress distribution and deformation characteristics of steel members and infilled concrete within and around the connection panels. The results indicated that the stress transfer mechanism was governed by the combined action of the vertical bearing resistance of the infilled concrete around the split-beam

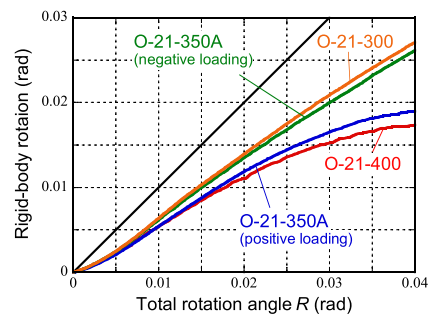
TABLE 4 | Flexural resistance around connection panels due to bearing action.

Specimens	Loading direction	f_c (mm)	x_U (mm)	x_L (mm)	${}_{tw}M_b$ (kNm)	${}_{Uf}M_b$ (kNm)	${}_{Lf}M_b$ (kNm)	M_b (kNm)	${}_bM_p$ (kNm)
O-60-300		72.8	78.3	78.3	122.9	11.1	0.0	134	163
O-60-400		72.8	149.8	149.8	122.9	40.8	40.8	204	163
O-60-350A	+	71.6	78.3	149.8	120.8	11.0	40.1	172	164
	–	71.6	78.3	149.8	120.8	0.0	40.1	161	164
O-21-300 ^[7]		31.8	78.3	78.3	53.7	4.9	0.0	58.5	169
O-21-400 ^[7]		33.1	149.8	149.8	55.9	18.6	18.6	93.0	169
O-21-350A	+	27.8	78.3	149.8	46.9	4.3	15.6	66.8	162
	–	27.8	78.3	149.8	46.9	0.0	15.6	62.5	162

Abbreviations: ${}_{Lf}M_b$, flexural resistance due to bearing of the lower flange; M_b , total flexural resistance arising from bearing stresses; ${}_{tw}M_b$, flexural resistance due to bearing of the column tube wall; ${}_{Uf}M_b$, flexural resistance due to bearing of the upper flange; x_L , embedded length of lower flange; x_U , embedded length of upper flange.



A



B

FIGURE 17 | Relationship between rigid-body rotation of split-beam and total deformation: (A) $F_c = 60$ MPa and (B) $F_c = 30$ Mpa.

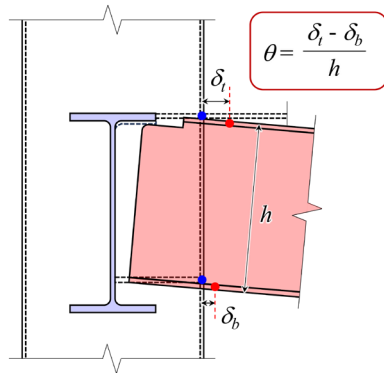


FIGURE 18 | Rigid-body rotation due to pull-out of the split beam.

flange and the horizontal bearing resistance transmitted through the tube wall.

3. Rigid-body rotation caused by pull-out of the split-beam flange decreased when a larger flange-bearing area was provided within the connection panel and higher-strength concrete was used. Under such conditions, subsequent deformation could be attributed to plastic rotation at the beam end rather than to additional rigid-body rotation.

These findings advance the understanding of the stress-transfer mechanism and deformation characteristics of split-beam connections in circular CFST beam-to-column connections. Although the rotational stiffness of the proposed connection is an important issue for structural analysis and practical design, its evaluation remains a subject for future study.

Acknowledgments

This research was funded by the Japan Society for the Promotion of Science (JSPS) Grant-in-Aid for Scientific Research (C) [grant number: 23K04120]. We would like to thank Editage (www.editage.jp) for English language editing.

Funding

Japan Society for the Promotion of Science (23K04120).

Conflicts of Interest

The authors declare no conflicts of interest.

Data Availability Statement

The data that support the findings of this study are available on request from the corresponding author. The data are not publicly available due to privacy or ethical restrictions.

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Supporting Information

Additional supporting information can be found online in the Supporting Information section. **Figure S1:** Specimen dimensions: (A)

Specimen O-60-300, (B) Specimen O-60-400, and (C) Specimen O-60-350A and O-21-350A. **Figure S2:** Joint details: (A) Specimen O-60-300, (B) Specimen O-60-400, and (C) Specimen O-60-350A and O-21-350A. **Figure S3:** Loading apparatus. **Figure S4:** Relationship between shear force and rotation angle: (A) Specimen O-60-300, (B) Specimen O-60-400, (C) Specimen O-60-350A, and (D) Specimen O-21-350A. **Figure S5:** Minimum principal stress distribution of infilled concrete (Specimen O-21-400, $R=0.02$ rad) [7]. **Figure S6:** Comparison of the relationship between shear force and rotation angle [7]: (A) O-300 series, (B) O-400 series, and (C) O-350A series. **Figure S7:** Stress distribution on the steel tube column surface in the axial and circumferential directions during positive loading (first cycle at $R=0.005$ rad): (A) Specimen O-60-300, (B) Specimen O-60-400, (C) Specimen O-60-350A, (D) Specimen O-21-300 [7], (E) Specimen O-21-400 [7], and (F) Specimen O-21-350A. **Figure S8:** Stress distribution on the steel tube column surface in the axial and circumferential directions during negative loading (first cycle at $R=-0.005$ rad): (A) Specimen O-60-300, (B) Specimen O-60-400, (C) Specimen O-60-350A, (D) Specimen O-21-300 [7], (E) Specimen O-21-400 [7], and (F) Specimen O-21-350A. **Figure S9:** Strain gauge positions on the column tube surface above and below the I-section beam. **Figure S10:** Example of finite element meshing (Specimen O-60-350A). **Figure S11:** Stress-strain curve of infilled concrete. **Figure S12:** Comparison between experimental and analytical behaviors: (A) Specimen O-60-350A (positive loading), (B) O-60-400 (positive loading), and (C) Specimen O-21-350A (negative loading). **Figure S13:** von Mises stress distribution on the steel tube and beam ($R=0.02$ rad). **Figure S14:** Local deformation around the tension-side flange at $R=0.02$ rad (Specimen O-60-300, deformation magnified three times). **Figure S15:** Stress distribution of infilled concrete ($R=0.02$ rad): (A) $F_c=60$ MPa, (B) $F_c=30$ MPa. **Figure S16:** Schematic of the resisting mechanism: (A) O-300 series, (B) O-400 series, (C) O-350A series (positive loading), and (D) O-350A series (negative loading). **Figure S17:** Relationship between rigid-body rotation of split-beam and total deformation: (A) $F_c=60$ MPa and (B) $F_c=30$ MPa. **Figure S18:** Rigid-body rotation due to pull-out of the split beam.