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EXPERIMENTAL STUDY ON CONTINUOUS BEAM TYPE SQUARE CFST BEAM-TO-COLUMN CONNECTION

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Concrete-filled steel tubular (CFST) columns are used widely as building structure components because of their good performance. It can be complex, however, to connect beams into CFST columns, especially when rigid beam-to-column connections are required. In order to simplify the fabrication requirements and cost of these connections, a continuous beam type connection through the column is proposed. This can develop the full plastic moment capacity of the beam. In this paper, the experiment testing results of two types of CFST beam-to-column connection specimens are presented and the connection performance and load transfer mechanisms are examined. The continuous beam type connection performance is compared to that of traditionally used through-diaphragm type of connection, which is more complex and demanding to fabricate. Both specimens showed good seismic performance, exhibiting stable hysteretic behavior with high energy absorption. The maximum experimental strength exceeded the calculated value. For a continuous beam type specimen, there is the potential for the column flange to be affected by the local deformation from the steel beam under bending induced beam tension. However, such effects were not observed in practice until late in the test after extensive beam plastic action had occurred.

Keywords: Concrete-filled steel tube, Connection performance, Energy absorption, Fillet welding, Full plastic moment capacity, Ultimate strength.

1 INTRODUCTION

Concrete-filled steel tubular (CFST) columns are used widely as building structure components because of their good structural performance under earthquake, fire and gravity loading and their ease of construction. Nevertheless, connecting beams into CFST columns can be especially complex when rigid beam-to-column connections are required. CFST beam-to-column connections are generally designed as thorough-diaphragm type, outer-diaphragm type, and inner-diaphragm type. Design formulas for these three types of beam-to-column connection are given in AIJ CFT Recommendations (Architectural Institute of Japan 2008) and in similar recommendations for New Zealand.

To simplify the fabrication requirements and costs of these connections, a continuous beam type connection has been proposed for CFST beam-to-column connections that, uses I-section beams passing through CFST columns (Azizinamini and Prakash 1993). This continuous beam type connection can easily develop the full plastic moment capacity of the steel beam and considerably simplify the design of CFST beam-to-column connection. For circular CFST columns, load transfer mechanism and design formulas are discussed, but few examples exist for square columns for CFST beam-to-column connections (Azizinamini and Schneider 2004, Schneider and Alostaz 1998).

As described herein, the experiment testing results of two types of square CFST beam-to-column connection specimens are presented and the connection performance and load transfer mechanisms are examined.

2 METHODOLOGY

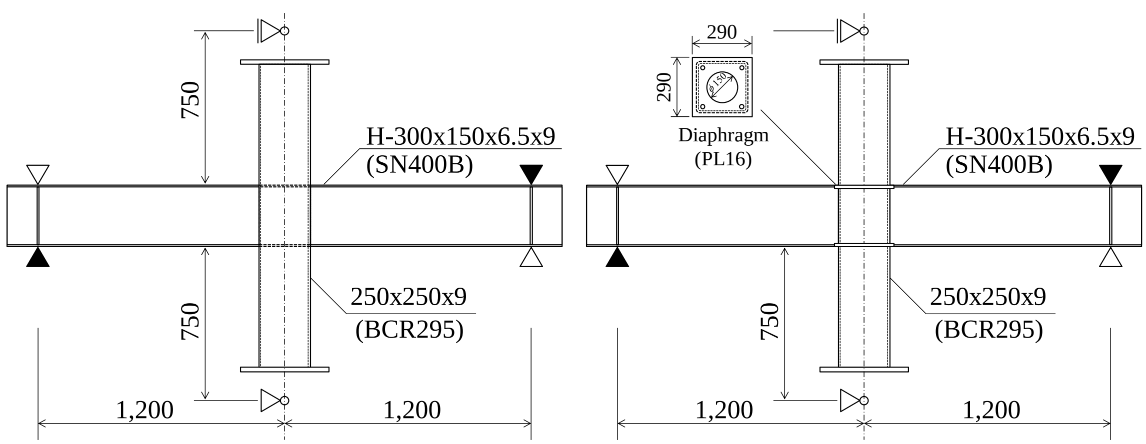
2.1 Specimens

The dimensions of the specimens are presented in Figure 1. Specimens are cruciform shape and simulating the square CFST beam-to-column connection. Two specimens of 1) Continuous beam type specimen and traditionally used 2) Through-diaphragm type specimen have been planned and designed so that beam yields prior to other parts (See Table 1).

Continuous beam type specimen required an I-section beam slot to be cut in column steel tube by plasma within 1mm tolerance. After putting the I-section beam through the slot, the I-section beam and outer faces of the column were fillet-welded all round.

A through-diaphragm type specimen has been designed according to AIJ CFT Recommendations. Connections of this type are more complex to design and demanding to fabricate.

The standard tensile and compressive tests were conducted for steel and concrete to ascertain the mechanical properties of the materials used. The measured results are presented in Tables 2 and 3. The relation between the concrete material age and strength are portrayed in Figure 3.



(a) Continuous beam type. (b) Through diaphragm type.

Figure 1. Dimension of specimens.

Table 1. Dimension of specimens.

Specimen	Column	Beam	Diaphragm	Strength of Concrete (MPa)
Continuous Beam Type	250x250x9 (BCR295)	H-300x150x6.5x9 (SN400B)	-	87.1
Through Diaphragm Type			PL16 (SN400B)	84.7

Table 2. Mechanical properties of steel.

		${}_sE$ (GPa)	f_y (MPa)	f_u (MPa)	Elongation (%)
Beam	I-section flange	203	328	474	29.4
	I-section web	203	350	472	23.6
Column	Steel Pipe	188	343	413	27.2
Diaphragm		208	277	437	30.7

${}_sE$: Young's modulus of steel, f_y: yield stress of steel, f_u: tensile strength of steel

Table 3. Material age and strength of concrete.

Age (days)	${}_cE$ (GPa)	f_c' (MPa)	ε_0	f_t (MPa)
7	-	70.7	-	-
14	-	71.3	-	-
21	-	74.6	-	-
28	38.5	83.1	0.00278	-
53	40.5	84.7	0.00276	4.92
74	41.0	89.4	0.00259	4.97
97	39.3	87.1	0.00275	5.18

${}_cE$: Young's modulus of concrete, f_c': compressive strength of concrete, ε_0: strain at compressive strength, f_t: split tensile strength

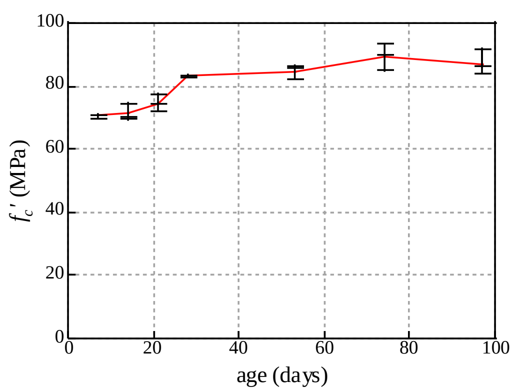


Figure 2. Change of concrete strength.

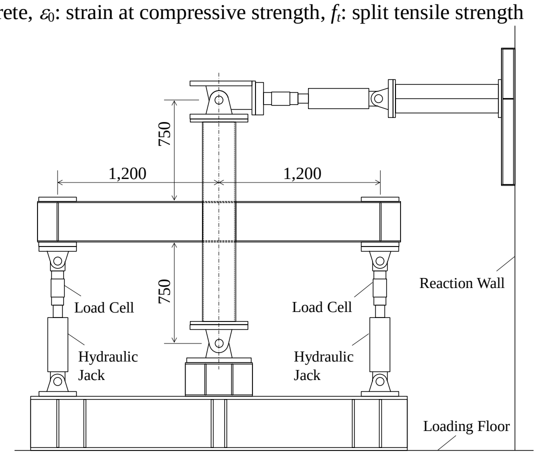


Figure 3. Experimental Setup.

2.2 Loading Method

The top end of the specimens was supported by hinge and roller, the bottom end was supported by hinge. Shear force was added by hydraulic jacks set at both beam ends (See Figure 3). Stiffening devices for the displacement to the out-of-plane direction were not set. Loading was controlled by rotational angle and the targeted rotation angles were 0.005rad, 0.01rad, 0.02rad, and 0.03rad. Rotation angle R is calculated from the relative vertical displacement of both beam ends and the distance between loading points.

3 EXPERIMENTAL RESULTS

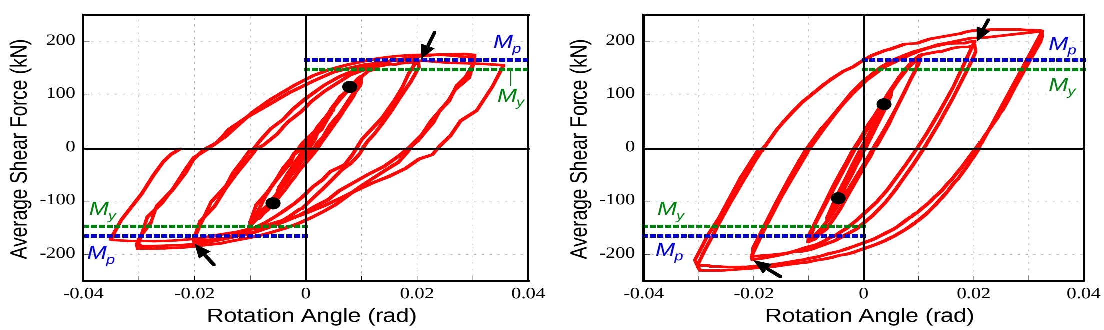
3.1 Shear Force and Rotation Angle Relation

Figure 4 presents the measured average shear force - rotation angle relationships. The measured behavior is shown as red solid lines. Black circles show the point at which the beam flange began yielding. Arrows indicate points at which local buckling was observed at the I-section beam flange. Green and blue lines show shear forces calculated using yield flexural strength and ultimate flexural strength. Table 4 presents experimental data and calculated data.

Both specimens showed good seismic performance exhibiting stable hysteretic behavior with high energy-absorbing capacity. The maximum experimental strength was observed at 0.03rad in each specimen. Regarding the continuous beam type specimen, the experimental value exceeded a calculated value by 10%. For the through diaphragm specimen, it exceeded the calculated value by 40%.

Table 4. Test results.

Specimen		Yield Strength (kN)			Ultimate Strength (kN)			Initial Rigidity (kN/rad)		
		Cal	Exp	Exp/Cal	Cal	Exp	Exp/Cal	Cal	Exp	Exp/Cal
Continuous Beam Type	+	147	114	0.78	165	177	1.07	17.3	12.6	0.73
	-		-105	-0.71		-189	-1.14			
Through Diaphragm Type	+		81	0.55		223	1.35	17.4	16.5	0.95
	-		-95	-0.65		-231	-1.39			



(a) Continuous beam type.

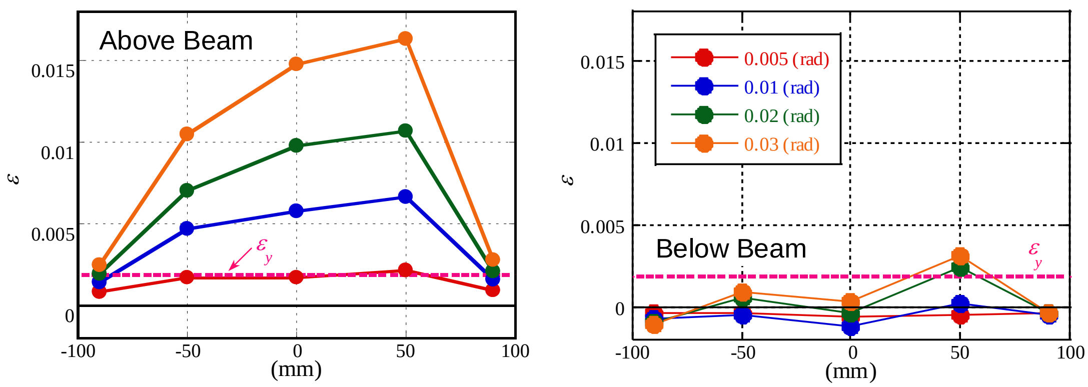
(b) Through diaphragm type.

Figure 4. Shear force – rotation angle relation.

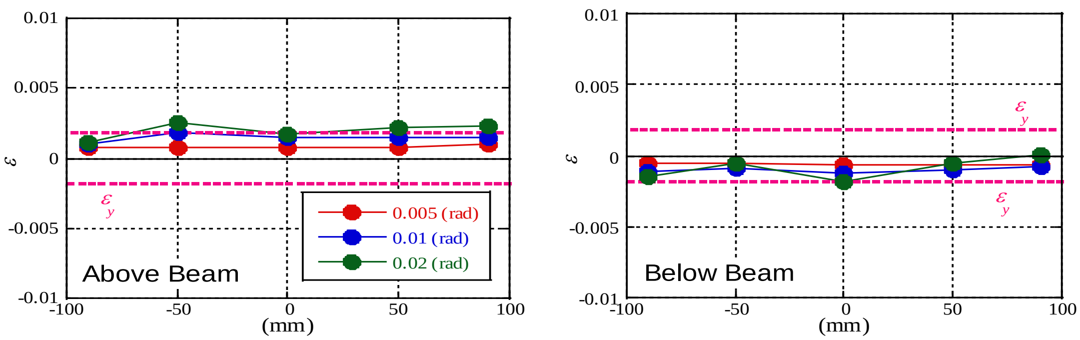
3.2 Strain Hysteresis

Figure 5 presents the strain distributions at the column flange above and below the steel beam. They were put in the position of 25mm away from the surface of the steel beam flange. For the continuous beam type specimen, the strain reached the yield strain at 0.005rad rotation angle. It rose thereafter. Parts for which the strain rose were only those just above and below the beam. The strain at the edge remained a small value. However, the strain value of the through diaphragm type specimen remained small. This tendency is the same as the range of minus rotation angle. Therefore, the column flanges of the continuous beam type specimen might be affected by the local deformation from the steel beam under bending induced beam tension.

However, such effects were not observed in practice until late in the test after extensive beam plastic action had occurred. Local deformation of the column flange above or below the I-section beam becomes large in a tensile region but remains a small value in a compression region, which indicates that the filled concrete contributes greatly to the transmission of the compressive load at the connection panel without a diaphragm plate.



(a) Continuous beam type.

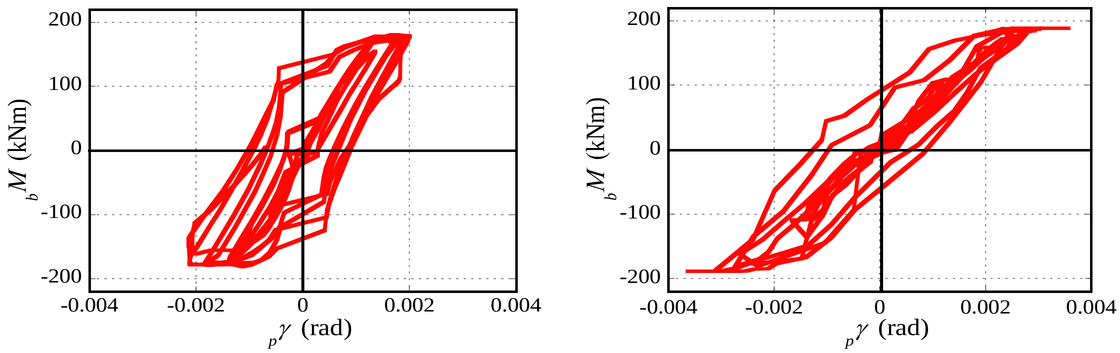


(b) Through diaphragm type.

Figure 5. Strain distribution at steel tube flange.

3.3 Shear Deformation of Panel Zone

Figure 6 shows the relation between the bending moment from the steel beam, ${}_bM$, and the panel shear deformation ${}_p\gamma$. The bending moment from the I-section steel beam was calculated using the strain gauge put in the position of 150mm away from the column surface. The shear deformation of the panel was calculated from the rosette strain gauge value put at the center of the panel web. Shear deformation of the panel of the through diaphragm type specimen was greater than that of the continuous beam type specimen. The rosette gauge value is only the local strain value. Therefore, it is considered that the overall panel deformation could not be measured in the shear deformation of the continuous beam.



(a) Continuous beam type. (b) Through diaphragm type.

Figure 6. $_bM - \gamma$ relation.

4 CONCLUDING REMARKS

1) Both specimens showed good performance with stable behavior and high energy-absorbing capacity. The maximum experimental strength exceeded the calculated value.
2) For the continuous beam type specimen, the column flange might be affected easily by local deformation from the steel beam under tensile conditions. However, local buckling of the steel column flange was not observed until the end of loading. Filled concrete contributes to transmission of the compressive load at the connection panel.

Acknowledgments

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