2197

BEHAVIOR OF BEAM-TO-COLUMN CONNECTION OF CFT COLUMN SYSTEM

Toshiaki FUJIMOTO[1], Eiichi INAI[2], Makoto KAI[3], Koji MORI[4], Osamu MORI[5] And Isao NISHIYAMA[6]

SUMMARY

As part of the U.S.-Japan Co-operative Earthquake Engineering Research Program on Composite and Hybrid Structures, to investigate the effects of material strength, connection configuration, geometry, axial load and loading direction on the shear strengths of beam-to-concrete filled steel tubular column (CFT) connections using high strength concrete and steel, eleven CFT connection specimens were tested under cyclic loading. This paper presents the test results of cyclic load-deformation behavior of CFT beam-to-column joint panels, and discusses the shear strength and the deformation capacity. Calculation methods of CFT joint panel are obtained from the Standard for Structural Calculation of Steel Reinforced Concrete Structures (SRC), 1987 edition, published by AIJ [3]. The yield shear of the CFT joint panel as described in AIJ-SRC is used for allowable stress design in moderate earthquake. For limit state design in severe earthquake, the member's ultimate strength is taken as 1.2 times the design yield strength. The elastic behavior of most specimens conformed well with the SRC design standard, disregarding their differences in material strengths, connection types, or loading directions. The exceptions were outer diaphragm joint specimen and variable axial load specimen, which had shown relatively large stiffness loss within the elastic range. All specimens reached an ultimate panel shear of 1.3 times their SRC design panel ultimate strengths and above. As the SRC design concept rests on the idea of forming plastic hinges at the beams during severe earthquake, equation should call for an ultimate-to-yield strength multiplication factor that insures the formation of such failure mechanism. The current AIJ-SRC recommendation is smaller than the experimentally obtained value and thus capable of safeguarding this design criterion for CFT connections made from both ordinary strength and high strength materials.

INTRODUCTION

This paper discusses the seismic behaviour of CFT beam-to-column connections used in buildings. The main object of this study is to especially expand the application scope of the CFT column system by utilising higher material strength both for the filling concrete and the steel tube than those applied in the present design standard. As part of the U.S.-Japan Co-operative Earthquake Engineering Research Program on Composite and Hybrid Structures, eleven CFT connection specimens were tested under cyclic loading [1]. These specimens were made from high strength concrete and steel. Moreover, the influence which connection configuration, geometry, axial load and loading direction affect on behaviour in the CFT beam-to-column connections is examined in this study. The load-deformation relationship of the joint panel zone of these CFT beam-to-column connections such as initial modules of elasticity, yield strength, ultimate strengths and deformation capacities were also examined.

[1] *Research Centre, ANDO Corporation, Saitama, Japan, E-mail: toshiaki.fujimoto@nifty.ne.jp*
[2] *Faculty of Engineering, Yamaguchi University, Yamaguchi, Japan, E-mail: inai@kde.yamaguchi-ac.jp*
[3] *Technical Research Institute, MATSUMURA GUMI Corporation, Hyogo, Japan, E-mail: kai@matsumura-gumi.co.jp*
[4] *Technical Research Institute, Asanuma Corporation, Osaka, Japan, E-mail: mori@tri.asanuma.co.jp*
[5] *Technical Research Institute, TOYO Corporation, Ibaraki, Japan, E-mail: o_mori@kb3.so-net.ne.jp*
[6] *Building Research Institute, Ministry of Construction, Ibaraki, Japan, E-mail: isao@kenken.go.jp*

SPECIMENS

Summaries of all the test specimens are shown in Table 1 and Figure 1. Among the eleven specimens tested, seven of them have square cross sections (specimens R1 to R6); the other four have circular cross sections (specimens C1 to C4). There are three specimens made into the shape of exterior joint of a building structure, namely R5 and R5' in the square section specimens and C4 in the circular section specimen. The rest of the specimens all represent interior joints of the building structure. To properly simulate seismic situation, this difference in joint type is taken into consideration when axial load was applied to the CFT column during testing. For the interior joint specimens, a constant axial load, equivalent to 20% of the compressive strength of the joint panel zone, was through the experiment. On the other hand, exterior joint specimens R5 and C4 were subjected to a varying axial load, cycling from 30% of the axial tensile strength of the steel portion of joint, to 70% of the axial compressive strength of joint. However, exterior joint specimen R5' was subjected to constant axial load as same as interior joint specimens.

Specimens R1, R2 and R3 are designed to examine the effect of material strengths on the shear performance of the CFT joint panels. R1 is a prototype specimen, in which same grade of steel is used for the column, beam and joint panel, with a designed tensile strength of 590MPa. The concrete filling the column has a designed compressive strength of 90MPa. As for R2, concrete strength is different from the prototype. 40MPa class concrete is used. As for R3, steel strength is different from the prototype. 780MPa-class steel is used. All three specimens represent reduced-size, 2-Dimensional interior connections in a building structure, with beams built continuously through the columns at the joint (through diaphragm type). C1, C2 and C3 are just the circular column version of above-mentioned. Specimens R4 and R6 planned only square cross section. R4 varies from the prototype by adopting an outer diaphragm connection design. R6 is the 3-Dimensional type of the prototype, and is loaded 45 degree to the face of the joint panel.

For all specimens, the steel thickness of the joint panel zone is significantly smaller than that of column. This is an intentional design to ensure that shear failure would take place at the panel.

The square CFT columns were fabricated by welding together two pieces of channel section, which were cold formed from flat plate. Circular columns were cold formed from press bending. Steel beams of all specimens were welded wide flange section. Concrete was cast into all specimens by filling it from the hole on top of each column.

Table 1: Test Specimens Details (unit: mm)

Speci-men	Joint Type	Diaphragm	Steel Tube		Concrete *f'c*	Beam(H-shape) (*Fy*)	Story Height *H*	Span *L*
			Panel (*Fy*)	Column (*Fy*)				
R1	Interior	Through diaphragm	Square-250*4.58 (492MPa)	Square-250*12.2 (449MPa)	110MPa	250*250*9.04*12.0 (448MPa)	3000	2990
R2					54.4MPa			
R3			Square-250*4.72 (756MPa)	Square-250*12.2 (759MPa)	103MPa	250*250*9.16*12.1 (739MPa)	3120	3000
R4		Outer diaphragm	Square-250*4.58 (492MPa)	Square-250*12.2 (449MPa)		250*250*9.04*12.0 (448MPa)		
R5	Exterior	Through diaphragm	Square-160*3.08 (513MPa)	Square-160*9.02 (440MPa)	99.6MPa	160*160*11.9*16.2 (493MPa)	2000	1000
R5'								
R6	Interior		Square-250*4.58 (492MPa)	Square-250*12.2 (449MPa)	97.7MPa	250*250*9.04*12.0 (436MPa)	3050	3500
C1	Interior	Through diaphragm	Circle-280*4.64 (439MPa)	Circle-280*12.3 (442MPa)	98.4 MPa	250*250*9.04*11.9 (448MPa)	2500	3000
C2					49.1MPa			
C3			Circle-280*4.78 (730MPa)	Circle-280*9.14 (763MPa)	94.2MPa	250*250*9.16*12.1 (739MPa)	2990	3000
C4	Exterior		Circle-180*3.09 (448MPa)	Circle-180*9.08 (475MPa)	99.6 MPa	160*160*11.9*16.2 (493MPa)	2000	1000

Fy: Yield strength of steel ***f'c***: Compressive strength of concrete
L: distance between point of shear load application for interior joint specimens
distance from the point of shear load application to center of joint panel for exterior joint specimens
H: distance between column restrains

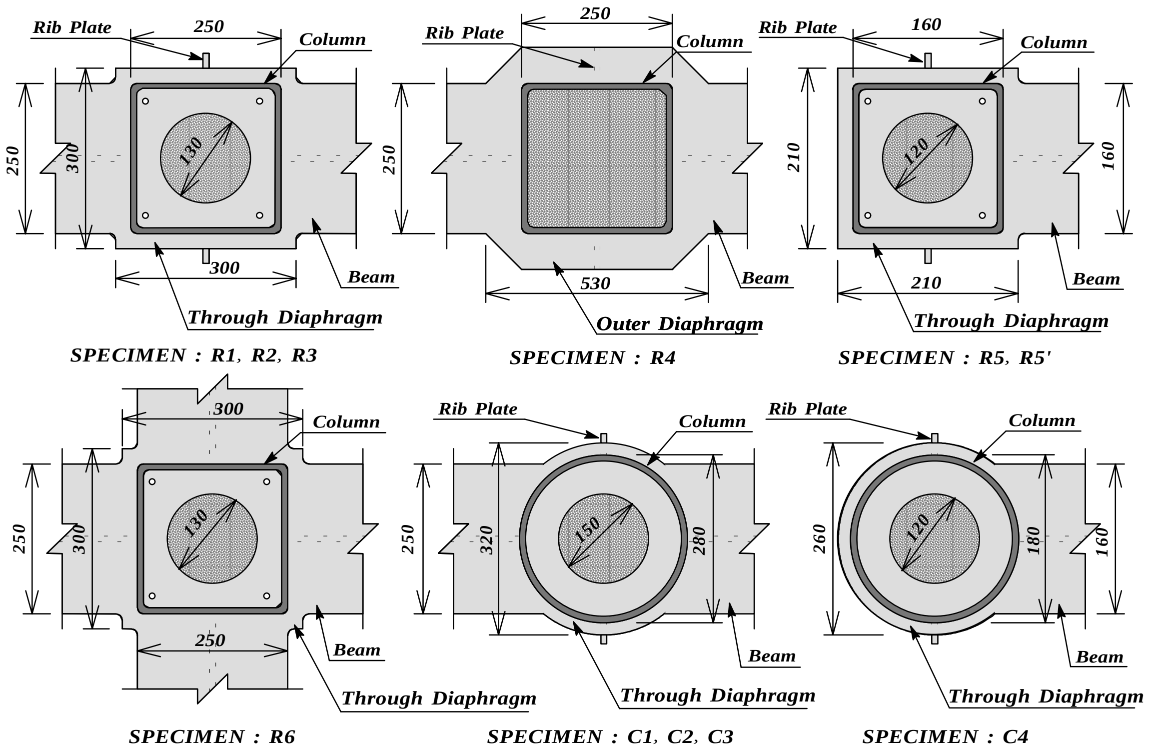


Figure 1: Details of Specimens

TESTING APPARATUS AND PROCEDURES

The loading condition is shown in Figure 2. For the interior joint specimens, constant axial load, equivalent to 20% of the compressive strength of joint panel zone ($0.2{}_pN_o$ where ${}_pN_o = F_y \cdot {}_sA + f'_c \cdot {}_cA$ based on the actual material strength and geometric properties of steel and concrete used), was applied by the middle actuator. The two side actuators applied cyclic loads by moving in opposite directions in order to achieve a prescribed story drift angle at each loading cycle. The loading scheme was aimed to stimulate the seismic behaviour of the CFT connection, and is summarised in Figure 3(a).

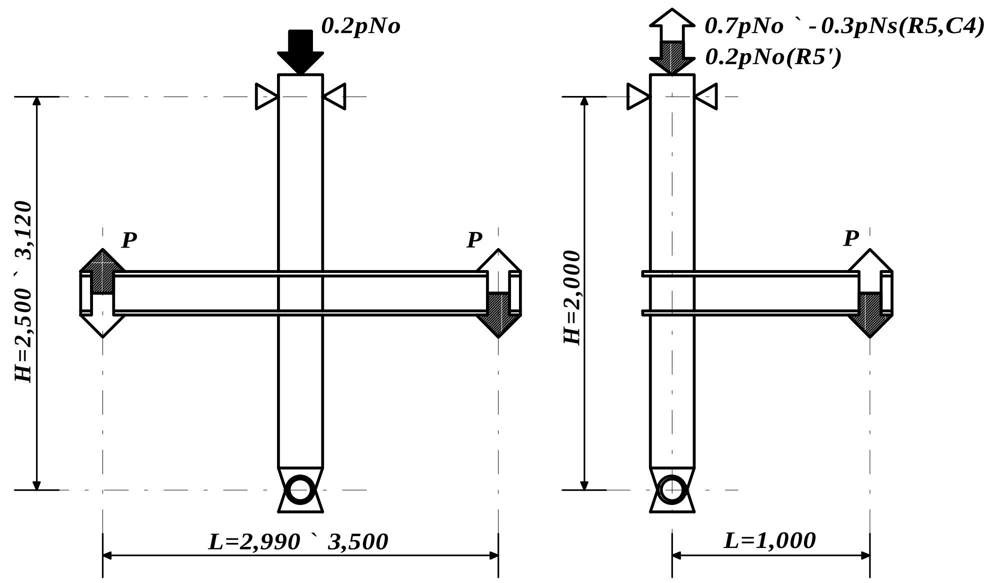


Figure 2: Loading Condition

In the case of the exterior joint specimen subjected to variable axial load, its intensity is 0.7_pN_o in compression in the positive side of the horizontal loading, and -0.3_pN_s (where ${}_pN_S = F_y \cdot {}_sA$) in tension in the negative loading side. When the shear load was zero, the axial load was switched (see Figure 3(b)).

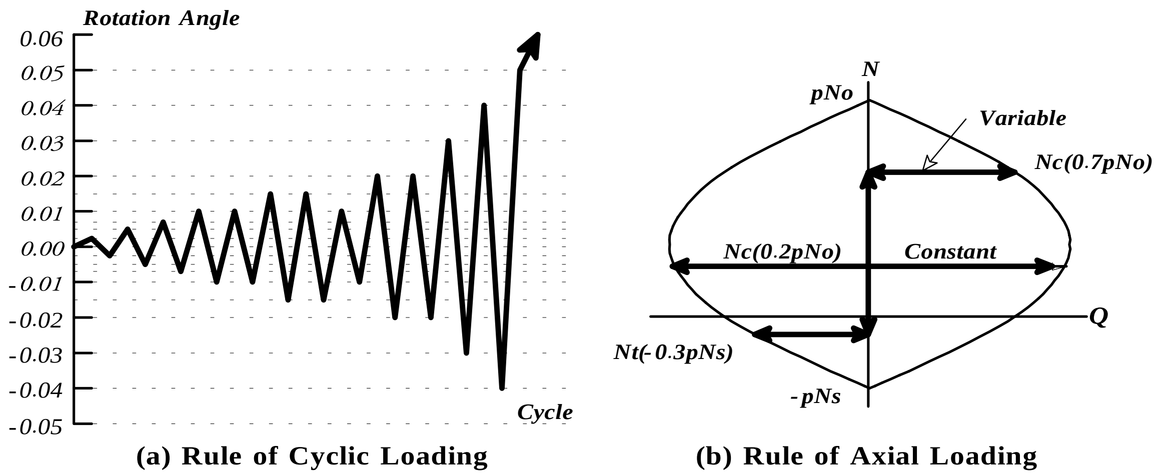


Figure 3: Loading Scheme

The story displacement and shear displacement of the connection and the deformation of the beam and the column were measured by displacement transducers. Moreover, the gage was affixed on the steel tube of column, the steel tube of connection, and the beam, and the strain of each part is measured.

CALCULATION

Calculation methods contained herein are obtained from the Standard for Structural Calculation of Steel Reinforced Concrete Structures (SRC), 1987 edition, published by AIJ [3].

Design Strength of joint panel

The short-term design yield shear of the CFT joint panel ${}_pQ_y$ can be calculated by the following equation:

$$ {}_pQ_y = \left(2\ {}_Jf_s \cdot {}_J\beta \cdot {}_cV + {}_sf_s \cdot {}_sV\right) / {}_{sB}d \tag{1}$$

where ${}_Jf_S$ =short term shear strength of concrete (kgf/cm^2); [$= 1.5 \times \min.\left(f'_c/30, 5 + f'_c/100\right)$]

${}_J\beta$ $= \min.\left(2 \cdot {}_sD/{}_{sB}d, 4.0\right)$ for circular column, $= \min.\left(2.5 \cdot {}_sD/{}_{sB}d, 4.0\right)$ for square column

${}_SD$: Diameter of steel tube (cm)

${}_{sB}d$: Center distance between upper and lower flanges of steel beam (cm)

${}_CV$: effective volume of concrete in the joint panel zone (cm^3); [$= {}_cA \cdot {}_{sB}d$]

${}_SV$: effective shear volume of steel in the joint panel zone (cm^3); [$= {}_sA \cdot {}_{sB}d/2$]

sfs : short term shear strength of steel (kgf/cm^2); [$= F_y/\sqrt{3}$]

The yield shear of the CFT joint panel, ${}_pQ_y$ is used for allowable stress design in moderate earthquake. For limit state design in severe earthquake, the member's ultimate strength is taken as 1.2 times the design yield strength (${}_pQ_u = 1.2 \cdot {}_pQ_y$). However, the high strength material used for this study is made outside the application range in this standard.

Comparison between Design Value and Experimental Result

To facilitate comparison between the AIJ-SRC design yield shear of the joint panel and the maximum shear load obtained from the experiment, ${}_pQ_y$ calculated from equation (1) must be transferred to the design shear at the column. To achieve so:

$$_{p}Q_{y} = n \cdot {}_{B}Q_{y} \cdot L' / {}_{sB}d - {}_{c}Q \quad (2)$$

where n : number of beams which place to panel [=2 for interior joint specimens, =1 for exterior joint specimens]
$_{B}Q_{y}$: design yield shear at the point of shear load application (at the beam ends) (kg)
L' : distance from point of shear load application to face of column (cm)
$_{C}Q$ =shear at the column ends (kg)

The column shear $_{c}Q$ is related to the beam shear $_{B}Q$ by the following equation:

$$_{c}Q = {}_{B}Q(L/H) \quad (3)$$

Therefore, rearranging terms in equation (2):

$$_{c}Q_{y} = {}_{p}Q_{y} / \eta \quad (4)$$

where $\eta = n \cdot H \cdot L' / (L \cdot {}_{sB}d) - 1$

Equation (4) gives the relationship between the column shear and the panel shear. The calculated $_{c}Q_{y}$ is then compared with experimentally obtained Q_{max}, the maximum of the average shear load applied at the two ends of the beam. Q_{max} is in turn taken as the average between these maximums, in absolute value.

EXPERIMENTAL RESULTS

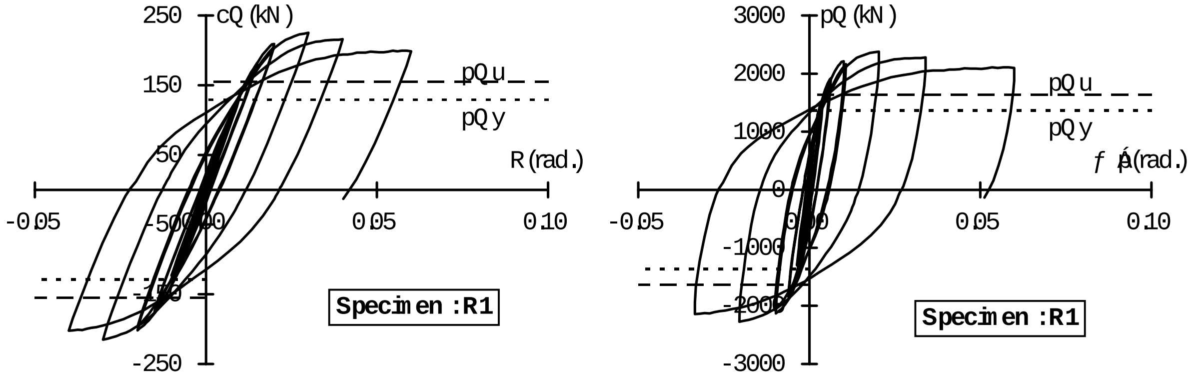
The test results are summarised in Table 2. The numerical value in the table is shown by the shear force of the column ends, $_{c}Q$ calculated from equation (3).

Experimental maximum strength of all specimens, $_{e}Q_{max}$ exceeded the design yield strength $_{p}Q_{y}$ and design ultimate strength $_{p}Q_{u}$. However, experimental maximum value does not reach calculation value of the CFT column and the steel beam. Therefore, it is thought that maximum strength of these specimens is decided by destruction of panel shear.

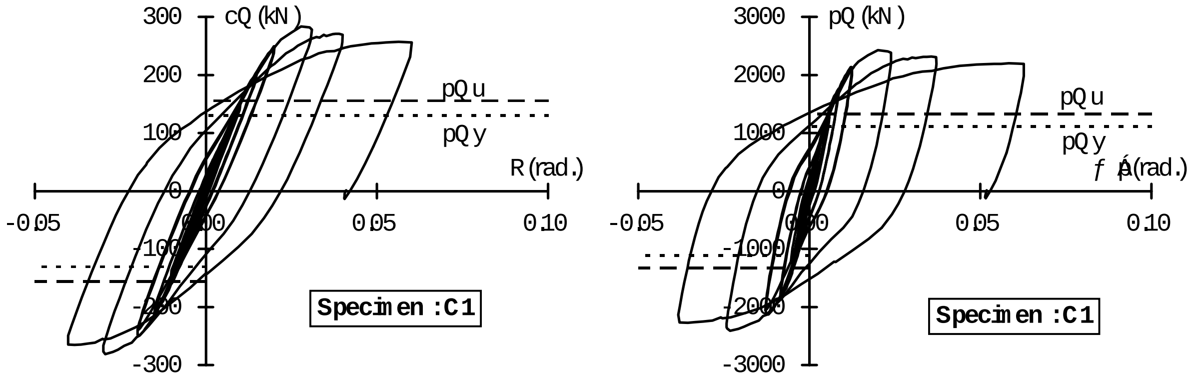
Table 2: Test Results (Unit: kN)

Speci-men	Axial load		Calculated value						Test result		$_{e}Q_{y}/_{p}Q_{y}$	$_{e}Q_{max}/_{p}Q_{u}$
			Column		Beam		Panel					
	N/pNo	*N*	Yield strength Q_{y}	Ultimate strength Q_{u}	Yield strength Q_{y}	Ultimate strength Q_{u}	Yield strength $_{p}Q_{y}$	Ultimate strength $_{p}Q_{u}$	Panel yielding load $_{e}Q_{y}$	Maximum applied load $_{e}Q_{max}$		
R1	0.2	1705	326	416	243	267	129	155	140	226	1.09	1.46
R2		1066	294	367			105	126	106	175	1.01	1.39
R3		1890	748	796	390	432	153	186	213	251	1.39	1.35
R4		1458	504	533	231	256	121	144	102	188	0.84	1.31
R5	0.7	2200	80	146	97.5	114	46.5	56.0	54	85.8	1.15	1.53
	-0.3	-258							-	88.5	-	1.58
R5'	0.2	643	142	180	97.0	113			50	90.9	1.07	1.62
R6	0.2	1486	232	425	231	253	120	144	167	225	1.39	1.56
C1	0.2	932	268	381	295	326	130	156	209	284	1.61	1.82
C2		1030	240	350			107	128	172	228	1.61	1.78
C3		1684	358	461	406	450	139	167	194	277	1.40	1.66
C4	0.7	2034	62	163	99	115	41.9	50.5	47	92.6	1.11	1.83
	-0.3	-231							-	93	-	1.84

Figure 4 shows the column shear $_{c}Q$ versus story drift angle R and the panel shear $_{p}Q$ versus panel drift angle r_{p} for the prototype of beam-to-column specimens, namely R1 and C1. The column shear $_{c}Q$ is derived from the experimentally measured beam shear $_{B}Q$ using equation (3) above. Similarly, the panel shear $_{p}Q$ is derived from the experimentally measured beam shear ${}_BQ$ using equations (2) and (3). In these figures, the SRC design yield shear of CFT joint panel by equation (1), ${}_pQ_y$ is shown in the dotted line. And the SRC design ultimate shear, ${}_pQ_u$ is 1.2 times the SRC design yield shear ${}_pQ_y$ is additionally shown in the broken line in these figures.



(a) Prototype of Square Joint Specimen



(b) Prototype of Circular Joint Specimen

Figure 4: Test Results of Load-Deformation Relations

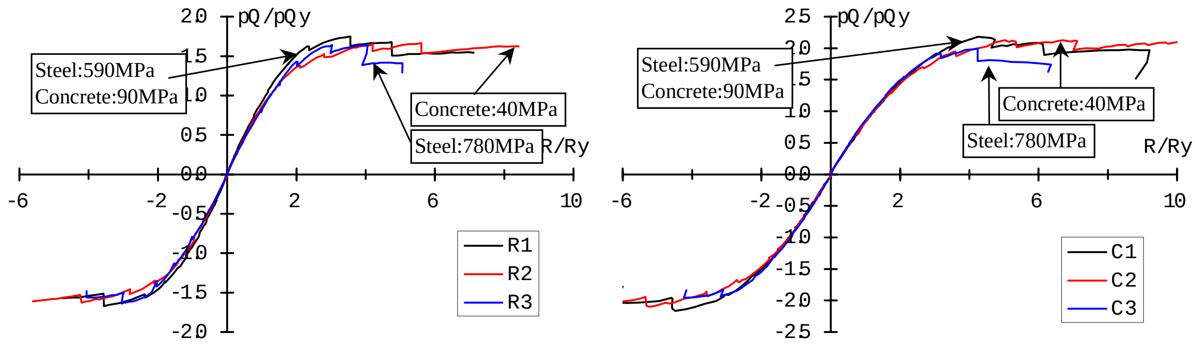
The deformation capacity was sufficiently large, and no drastic strength reduction was observed. In these specimens, as for the overall deformation, the deformation of the joint part is predominant. The deformation of the part of column and beam is little, and the behaviour is elastic. This tendency is similar concerning other specimens.

Figure 5 shows the normalised panel shear versus the normalised story drift angle for the specimens. The panel shear, ${}_pQ$ is normalised against the SRC design yield shear of CFT joint panel, ${}_pQ_y$. Similarly, the story drift angle measured directly during the experiment, R_{exp} is normalised against the story drift angle at SRC panel yielding R_y.

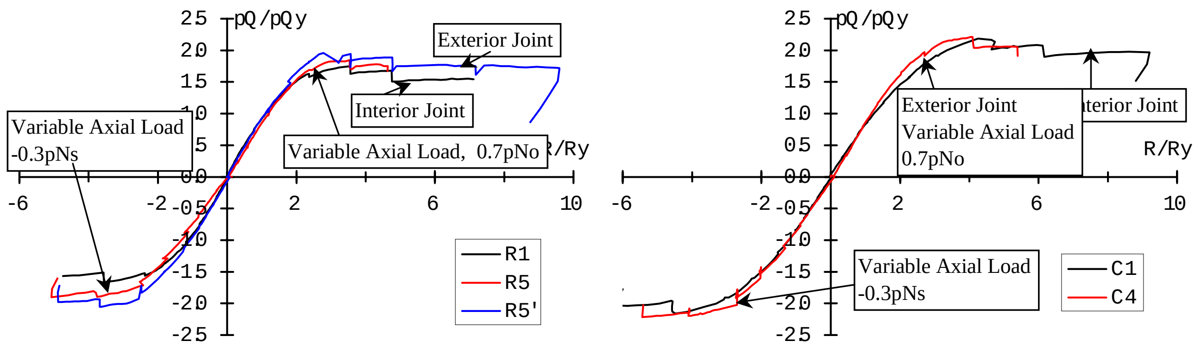
Generally, until the design yield strength ${}_pQ_y$ is reached, the experimental results for most specimens conform well to the initial stiffness, disregarding their differences in material strengths, connection types, axial loads, or loading directions (see Figure 5 (a) and (d)).

Some decrease in initial stiffness is observed in the elastic range, suggesting that local yielding and deformation of the materials around the joint panel zone could have happened. Specimen R4, which has adopted an outer diaphragm design, displays slightly larger stiffness loss as compared with prototype specimen, R1 (see Figure 5(c)). As anticipated, the outer diaphragm does not render the same confinement effect to the concrete core at the joint panel zone as that offered by the through diaphragm. Specimen R5, when subjected to the tensile axial load, displays slightly larger stiffness loss as compared with prototype specimens, R1 (see Figure 5(b)). The stiffness of this specimen is the middle of the design elastic stiffness of steel portion and sum of concrete portion and steel portion.

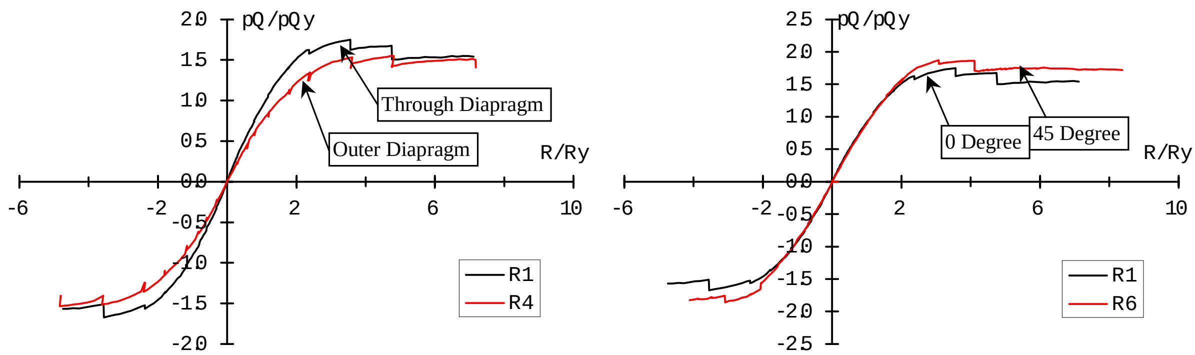
However, the deformation capacity of all specimens was sufficiently large, and drastic strength reduction was not observed. The rotation capacity of these specimens was exceeded 0.03 radian. Maximum strength of these specimens exceeded the design yield strength ${}_{p}Q_{y}$ and design ultimate strength ${}_{p}Q_{u}$. It is understood that the CFT joint panel shows very ductile behaviour from the above-mentioned.



(a) Effect of Material Strength



(b) Effect of Axial Load

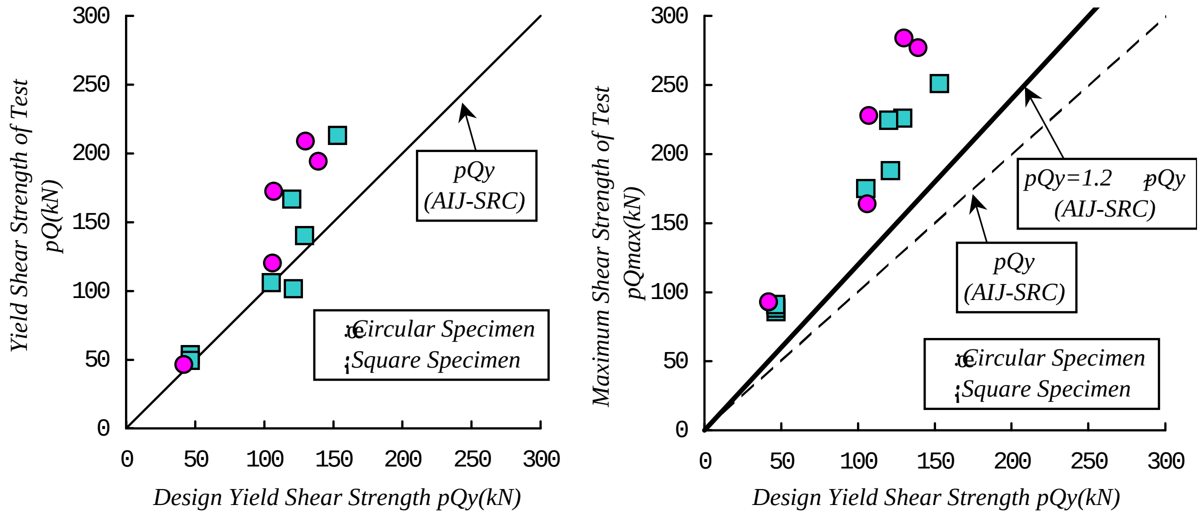


(c) Effect of Diaphragm Type **(d) Effect of Loading Direction**

Figure 5: Normalised Panel Shear vs. Normalised Story Drift Angle

Figure 6(a) shows the experimentally yield strength of panel shear versus the design yield strength for all beam-to-column specimens. The experimentally yield strengths and the design yield strength of beam-to-square column specimens show comparatively good agreement. The experiment value was a value from 0.84 to 1.39 times design yield strength. The experimentally yield strengths of beam-to-circular column specimens exceeded the design yield strength, and indicated 1.11 to 1.61 times the value.

Figure 6(b) shows the maximum strength of panel shear in the experiment versus the design yield strength for all specimens. An ultimate shear load of All specimens indicated larger than SRC design ultimate strength ${}_{p}Q_{u}$. Among this figure, the ones of circular column specimens display relatively higher values as compared with those of square column specimens, the former ranges from 1.96 to 2.22 while the latter 1.53 to 2.07. This observation is no more than another verification of the more promising shape possessed by circular cross section to generate hoop tension at high stress around the column steel wall, thus producing a continuous confining pressure to the concrete core. The lowest value within each group belongs to the specimen made of 780MPa high strength steel (R3 and C3).



(a) Yield Strength **(b) Ultimate Strength**

Figure 6: Design Strength vs. Test Result

As mentioned before, for limit state design in severe earthquake, the AIJ-SRC standard suggests multiplication factor of 1.2 for the design ultimate shear strength of CFT joint panel is recommended. Of interest is the fact that during severe earthquake, a moment resisting frame of a building structure is designed to form plastic hinges at the beams (the so-called strong column-weak beam concept) as a desirable failure mechanism. Thus, the multiplication factor of 1.2 actually serves as an insurance to allow for the achievement of the fully plastic bending at beam section. The experimental results, showing an ultimate to yield shear strength ratio larger than the standards' recommendations, prove that the standards' recommendations conform with this design concept for CFT connections made from both ordinary strength and high strength materials.

CONCLUSION

To investigate the effects of material strength, connection configuration, geometry, axial load and loading direction on the shear strengths of CFT beam-to-column joint panels using high strength concrete and steel, eleven CFT connection specimens were tested under cyclic loading.

1. The elastic behavior of most specimens conformed well with the SRC design standard, disregarding their differences in material strengths, connection types, or loading directions.
2. The exceptions were specimen R4 (outer diaphragm design at joint) and R5 (variable axial load), which had shown relatively large stiffness loss within the elastic range.
3. The ultimate panel shear strength is exceeded their SRC design panel ultimate strengths with all specimens.
4. The current AIJ-SRC recommendation is smaller than the experimentally obtained value and thus capable of safeguarding this design criterion for CFT connections made from both ordinary strength and high strength materials.

REFERENCES

1. Fujimoto, T. et al (1997), "Test Results of CFT Beam-to-Column Connection", Proceedings, *The fourth joint technical co-ordinating Committee Meeting on Composite and Hybrid Structures*, Monterey, California
2. Fujimoto, T. et al (1998), "Behaviour of beam-to-column Connection of CFT Column System", Proceedings, *The fifth joint technical co-ordinating Committee on Composite and Hybrid Structures*, Tokyo

Architectural Institute of Japan (1987), *AIJ Standard for Structural Calculation of Steel Reinforced Concrete Structures*