Through beam with bolted brackets connections for CFT column with steel beam

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ABSTRACT: Interior connections with steel beams and concrete filled tubular (CFT) columns were tested, under cyclic displacement controlled load. Square and circular steel tubular columns were considered. A new type of through beam connection, where the beam passes through the joint and connects with additional bolted brackets without using any welding between the beam and column was considered. The experiments demonstrated the capability of the through beam connection with bolted brackets to develop the full plastic flexural capacity of the beam. The connections exhibited ductile behaviour and the beam failure took place by formation of a plastic hinge in the beam away from the joint. The beams had inelastic rotational angles of 0.077 and 0.06 radians for the connection with square and circular CFT columns respectively, which were in excess of 0.04 radians as recommended by AISC (2002) for high seismic areas. The behaviour of the panel region was examined, by comparing calculated and measured shear capacities of the panel zone and it was found that the equations used for calculating the panel zone capacity were conservative and can be used for design. A simple analytical model was developed using RUAUMOKO-2D software in order to predict moment capacity of the connections. The analytical results matched well with the test results, and this demonstrates the ability of the proposed analytical model to simulate cyclic behaviour of through beam connection, very well. The proposed through beam with bolted brackets connections performed well and avoids site welding and hence are suitable for usage in high seismic areas.

1 INTRODUCTION

In high seismic risk areas such as California and Japan, steel-framed buildings have frequently been employed because of their excellent performances in terms of strength and ductility. Nevertheless, a large number of entirely unexpected severe brittle cracks of welded beam-to-column connections were found in the Northridge (1994) and Kobe (1995) earthquakes (Bertero et al, 1994, Kuwamura, 1998). The failures raised many questions regarding the validity of design and construction procedures used for these connections at the time. Since the earthquake, several extensive analytical and experimental studies have been conducted to investigate the various aspects believed to be associated with the failure observed in the pre-Northridge connection and to improve connection performance. For example, Schneider and Alostaz (1998) tested six large-scale specimens with different stiffening details. These details ranged from a very simple detail that attached the girder directly to the tube skin to a more rigid detail in which the girder was passed through the tube core. The connection detail that best simulated a rigid connection and exhibited good cyclic behaviour was the through beam connection detail. Azizinamini et al (2004) and Elremaily et al (2001) showed that this type of connections has high strength and plays an important role in buildings survival during a seismic event. Kimura and Matsui (2000) and Masuda et al (2000) investigated the performance of CFT column to H beam connections with vertical stiffener plates. Kato et al (1992); Morino et al (1992) and Fukumoto and Morita (2000) investigated the connections of internal and external diaphragm plates. Cheng and Chung (2003) investigated the connection details and shear strength in the panel zone of CFT through beam connections. These studies suggested that connections loading the skin of the steel tube only, can cause excessive deformation demands on the tube wall and connection components. Embedding

connection components into the concrete core alleviates high shear demand on the tube wall, which may improve the seismic performance of the connections. The through beam type connection detail prevents the transfer of large beam shear forces directly to the steel tube. This helps to prevent the steel tube from pulling away from the concrete core. The through beam type connection detail eliminates the need for welding thick connection elements to relatively thin steel tubes, which results in lower residual stresses than direct welding. However, the through beam type connection with site welding has not been favourable in construction practice because it requires considerable work in erection and extensive welding as well as it needs high tolerance in detailing.

Therefore, there is a need for studies on steel beam-CFT column connections that are practical, easy to assemble and provide good performance, where the region of yielding is moved away from the area of significant welding, to avoid the limiting possibility of premature weld fracture, and to avoid site welding, so that the construction quality and speed can be improved. The present study has a three-fold purpose:

- Investigating experimentally the seismic performance of through beam connections, where the beam passing through the joint and connected with additional bolted brackets without using any welding between the beam and the column.
- Examine the panel joint region of through beam connection.
- Developing a simple analytical model to simulate the response of the through beam type connection.

2 EXPERIMENTAL PROGRAM

2.1 Test specimens and configuration

The experimental program is composed of half-scale models of interior steel beam to square CFT column (SCFT) and circular CFT column (CCFT) subassemblies as shown in Figure 1. The design philosophy adopted in this study is to have a strong column; strong connection and weak beam. Applying this philosophy to connection required that all yielding would occur only in the beams while the connections, panel zones and column were designed to remain elastic throughout the testing.

The details of the test specimens are shown in Figures 2 and 3 respectively. These specimens consisted of steel beams passing through the column to represent an interior joint in a building. An opening in the shape of the steel beam but with 2 mm oversize was cut in the steel tube (column), to allow the girder to pass through the column. There was no welding between the steel beam and the column. This eliminates field welding, which is time-consuming and costly. A silicone layer was used to fill all the gaps on the outer surface, to prevent leaking of the fresh concrete and water. Flat bolted brackets and curved bolted brackets for specimens SCFT and CCFT respectively are shown in these figures. This bracket is similar to the cast steel Kaiser Bolted Bracket (KBB), pre-approved for special moment frame connections, by the ANSI/AISC 358 seismic provisions (2010). It reinforces the beam portion near the column, moving the location of the critical moment away from the column face.

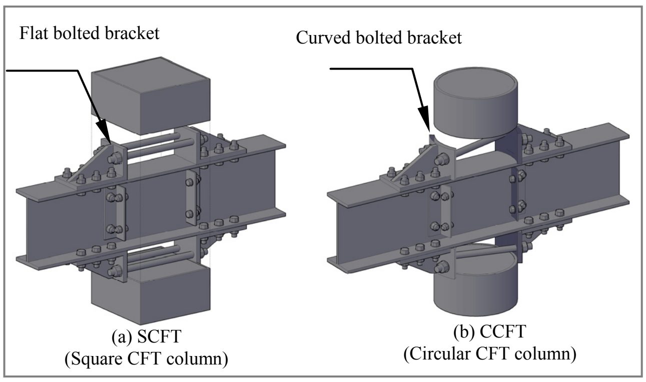


Figure 1. Test specimens with through beam type connections

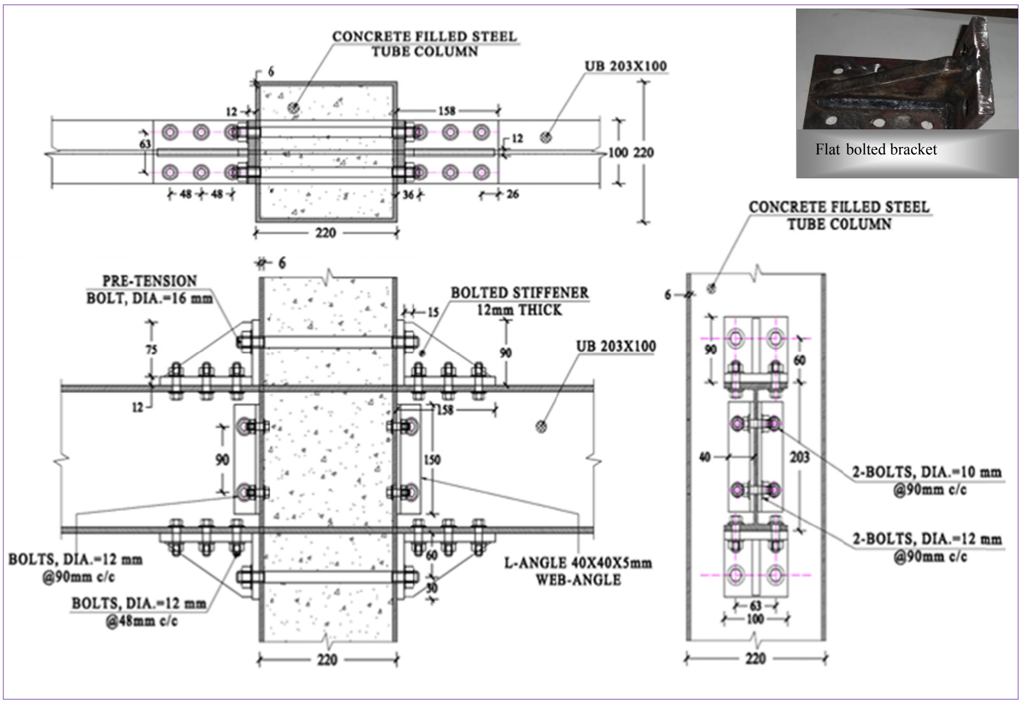


Figure 2. Details of specimen SCFT

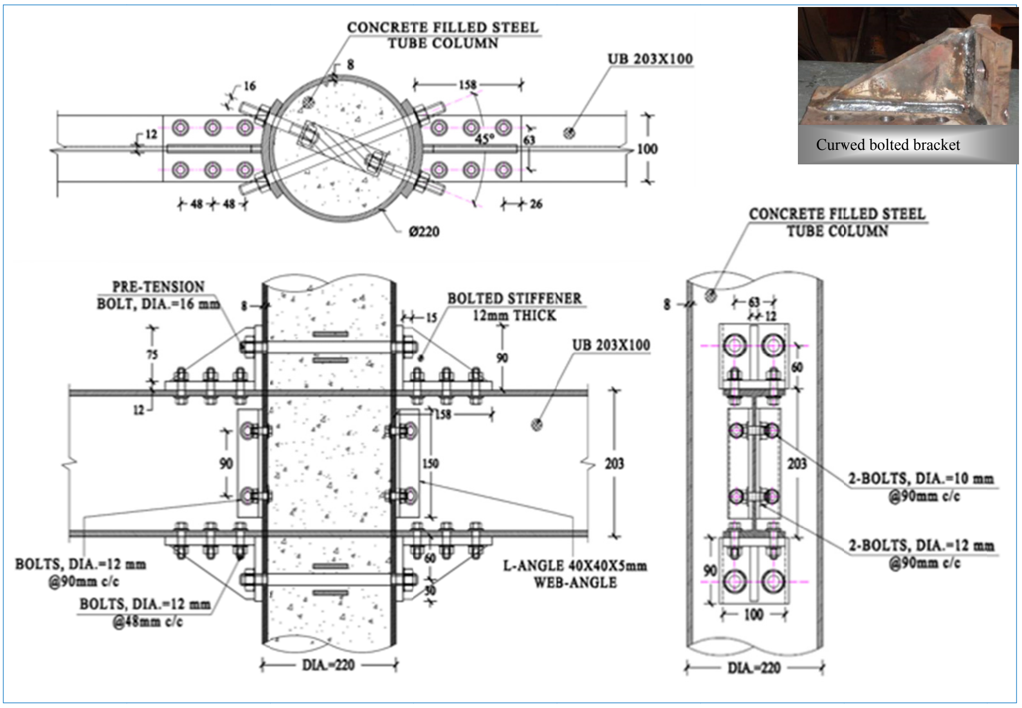


Figure 3. Details of specimen CCFT

2.2 **Test setup and loading history**

The test setup shown in Figure 4 was designed to test the interior beam-to-column joint models simulating seismic loading conditions. The specimens were tested under cyclic displacement controlled load, following a loading history consisting of a stepwise increase in the deformation cycles. Each loading step was defined by peak beam rotation, and by the number of cycles as shown in Figure 5. A six laser sensors were used to measure the beam vertical displacements. The shear deformation of the panel zone was calculated, using small linear variable differential transducers

(LVDTs) installed in “×” shape. A sixteen channel data acquisition system (DEWE-43) was used to monitor and control the displacement and force feedback signals. More details on the test set-up and on the experimental program as a whole, can be found in reference (Ikhlas et al 2013).

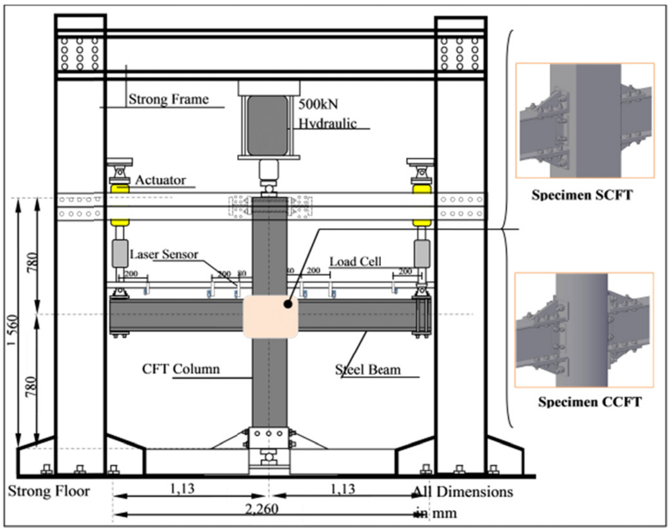


Figure 4. Test setup

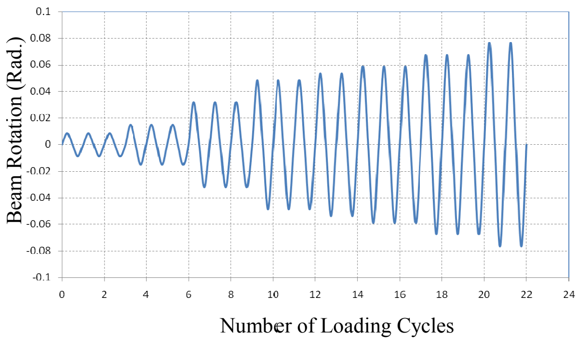


Figure 5. Cyclic loading history

3 MATERIAL TESTS

Procedures as per IS 1608 (2005) were followed to test the steel samples. Yield strength of the steel material and the concrete strength at the age of 28 days, and on the day of testing, are summarized in Table 1.

Table 1. Material strengths of the steel and concrete

Yield strength of the steel material					
No.	Sample		Elongation (%)	Yield strength (MPa)	Ultimate strength (MPa)
1	Beam section UB 203×100 (Grade 250)	Flange	38.0	293.80	451.93
2		Web	39.4	264.20	393.57
3	Square tube (Grade 300)		36.0	326.27	470.60
4	Circular tube (Grade 250)		21.3	285.91	447.95

Compressive strength at the day of test (MPa)		
	f_c'	f_{cu}
Specimen SCFT	34.38	41.37
Specimen CCFT	34.86	43.52

4 ANALYTICAL INVESTIGATION

Nonlinear finite element program RUAUMOKO-2D Software (Carr 2008) was used to create the model of the proposed new connection in order to predict the moment capacity of the connection. A model representing an interior frame connection was developed as shown in the Figure 6 using elements from the standard library. Frame element is used to represent the steel beams and CFT columns. Spring element is used to represent the rods in the bolted and flanges of the steel beam. Shear spring element used to represent the web of steel beam. The behaviour of the joint elements is

characterized by Ramberg-Osgood hysteresis loop.

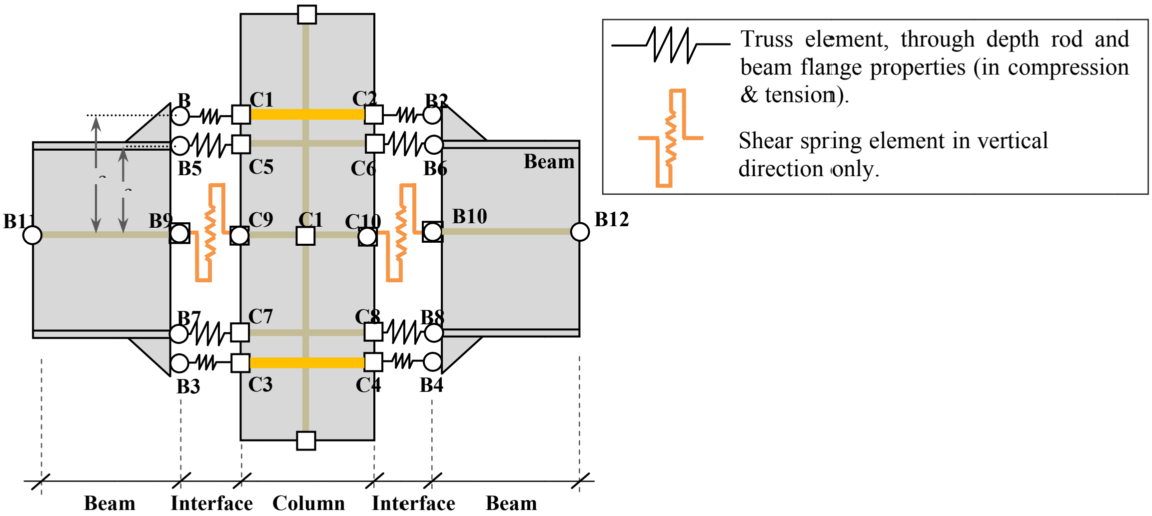


Figure 6 Computational model

5 RESULTS AND DISCUSSION

From the general behaviour of the experimental results, the specimens performed well with good ductility, and the plastic hinges were formed in the beam end near the column face. This behaviour is due to the strong column-weak beam design. The column and panel zone remained elastic during all stages of loading as shown in Figure 7.

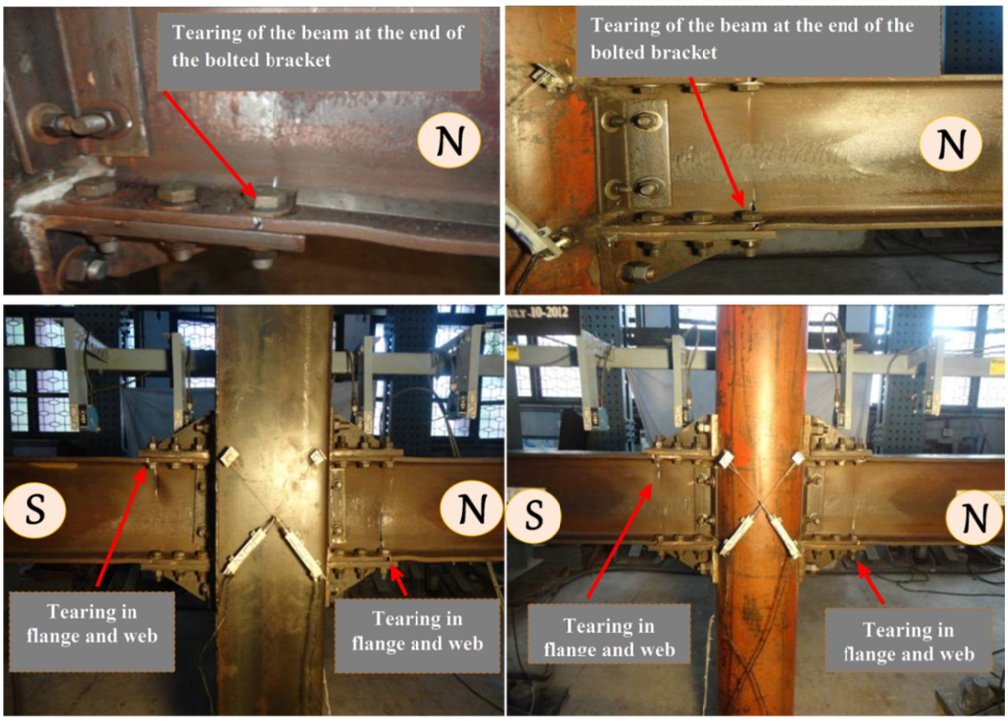
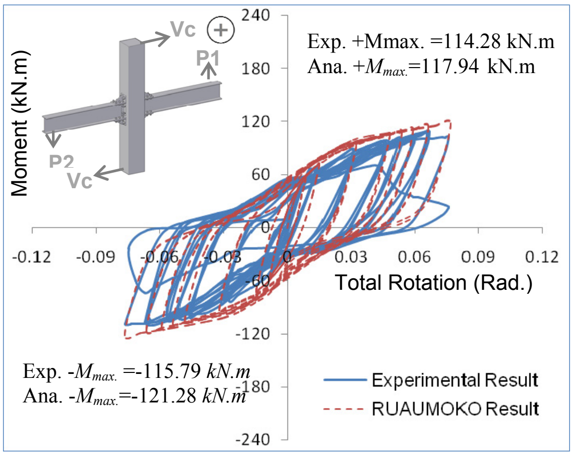


Figure 7. Photographs of test specimens SCFT and CCFT at the end of the test

The comparison of the hysteretic curves of the moment-rotation for both specimens (Figures 8 and 9) showed a satisfactory agreement between the numerical and experimental results. For specimen SCFT the maximum beam rotations of 0.077 radians were sustained successfully, in both directions of loading. For Specimen CCFT the maximum total beam rotation were 0.06 radians, in both directions of loading. It can be noted that the analytical model is reasonable in reproducing a similar hysteretic

response as the test results, though some differences of reloading stiffness exist.



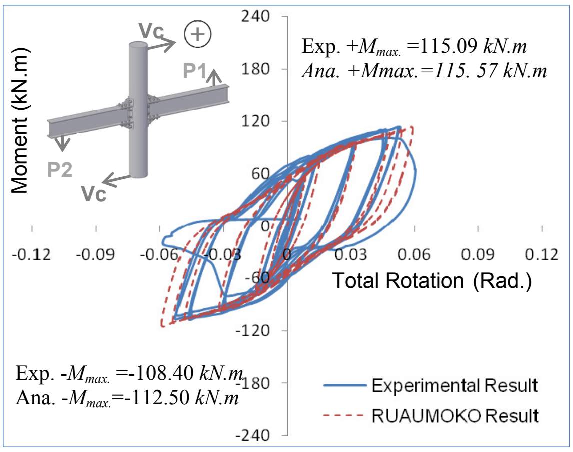


Figure 8. Beam moment vs. total rotation for specimen SCFT

Figure 9. Beam moment vs. total rotation for specimen CCFT

The comparison of the energy dissipation capacity for both specimens is shown in Figure 10. It can be found that the analytical energy dissipation capacity is similar to the experimental energy dissipation capacity for this type of connection, indicating that the model provided reasonable correlation with the experimental measurements. However, specimen SCFT exhibited larger energy dissipation capacity than the specimen CCFT since it had sustained a greater number of cycles. The comparison of the envelope curve is shown in Figure 11. This figure illustrates that the analytical envelope curves for both specimens are in closer correlation with the experimental measurements.

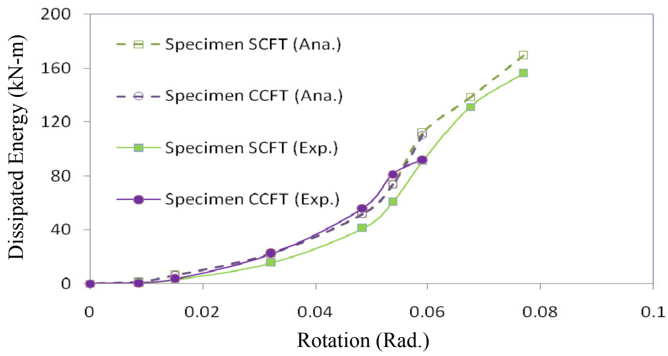


Figure 10. Dissipated energy vs. rotation curves for the test specimens

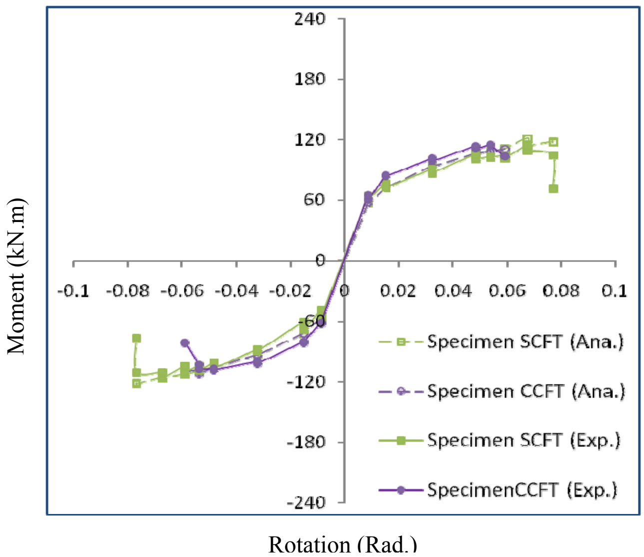


Figure 11. Comparison of envelope curves for the test specimens

6 PANEL JOINT

6.1 Joint shear deformation

To monitor the overall joint shear deformation in an average sense, two LVDTs were installed at the face of the joint in each specimen in "×" shape (Figure 12a). The joint shear strain was calculated using Eq. (1):

$$\gamma_j = \frac{\sqrt{b^2 + h^2}}{2.b.h}\left[\Delta_1 + \Delta_2\right] \tag{1}$$

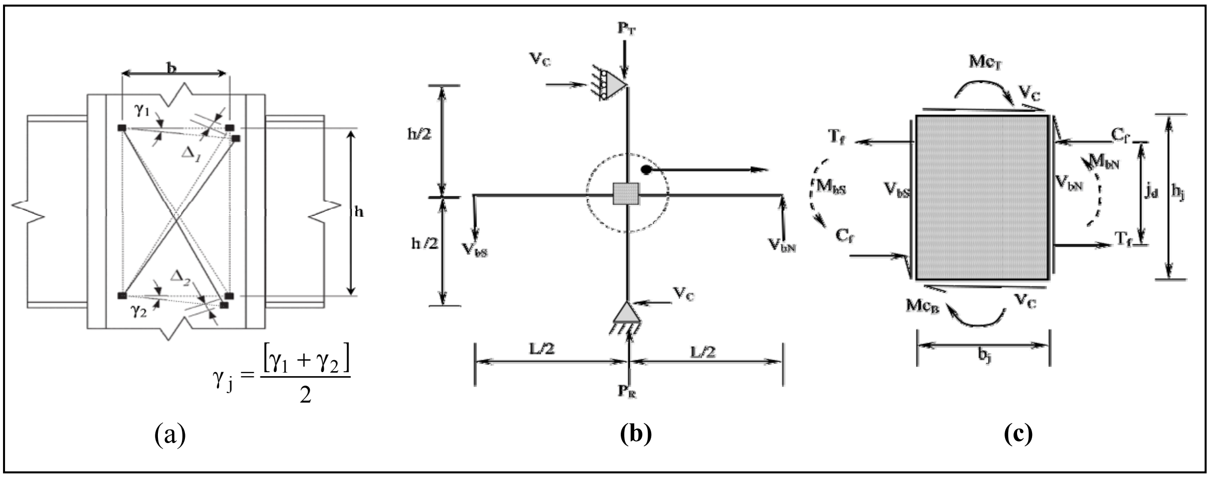


Figure 12. Joint shear deformation (a) Panel zone shear deformation measurement (b) typical interior beam-column joint test setup and (c) its joint panel

On the other hand the joint shear strength is evaluated, by studying the equilibrium of the horizontal forces on a horizontal plane at the mid height of the joint, as shown in Figure 12b and c. Assuming that the beam bending moment is carried entirely by the flanges, the tensile and compressive forces in the beam flange, T_f and C_f, are estimated as:

$$T_f = C_f = \frac{M_b}{j_d} \tag{2}$$

where M_b is the beam moment at the joint face; and j_d is the internal lever arm for calculating the moment (j_d=d-t_f).The effective horizontal shear force acting on the joint panel, V_{jh} is calculated using Eq.(3):

$$V_{jh} = (T_f + C_f) - V_C \tag{3}$$

where V_C is the column shear force and is estimated from the beam moments at the joint face using Eq.(4):

$$V_C(h) = V_{bS}\left[\frac{L}{2}\right] + V_{bN}\left[\frac{L}{2}\right] \tag{4}$$

The joint shear is expressed as follows:

$$V_{jh} = \left[\left(\frac{M_{bS}}{j_d}\right) + \left(\frac{M_{bN}}{j_d}\right)\right] - \left[V_{bs} + V_{bN}\right]\left(\frac{L}{2.h}\right) \tag{5}$$

M_{bN} and M_{bS} are the beam moments at the joint face, and are equal to $V_{bN} \times \left[(L/2) - (b_j/2)\right]$ and $V_{bS} \times \left[(L/2) - (b_j/2)\right]$ respectively. Substituting Eq. (5.5) in Eq. (5.4), the joint shear force is expressed as given in Eq. (6):

$$V_{jh} = [V_{bs} + V_{bN}]\left(\frac{L - b_j}{2.j_d} - \frac{L}{2.h}\right) \tag{6}$$

Where, L is the total length of the beam (includes south and north beam), bj is the width of the joint panel, and h is the total length of the top and bottom columns; (subscripts S and N refer to the South and North directions, respectively).

6.2 Evaluation of nominal shear capacity at the panel zone

The joint nominal shear force capacity is calculated, using Eq.(7).

$$V_n = V_{tn} + V_{cn} + V_{wfn} \tag{7}$$

The shear capacity of the steel tube (V_{tn}) is calculated using Eq. (8) (Krawinkler 1978) for the steel tube. For the concrete core the shear capacity (V_{cn}) is calculated based on ACI-ASCE 352 (1985) recommendations for reinforced concrete joints confined on all four vertical faces of the joint; since the joint in a CFT column is confined by the tube wall, it is reasonable to consider the same value as that recommended for confined joints and is presented in Eq. (9).

$$V_{tn} = A_{sht}\left[\frac{F_{yt}}{\sqrt{3}}\right] \tag{8}$$

$$V_{cn} = 1.99\sqrt{f_c'} \times A_{shc} \tag{9}$$

where F_{yt} and f_c' are the yield strength of the steel tube and concrete compressive strength respectively, while A_{shc} and A_{sht} are the horizontal effective shear area for the concrete core and the steel tube respectively. The effective shear area of the circular tube is given by $\pi\, d_c t_t / 2$, (Boresi et al 1993), and that of the rectangular tube is given by $2\,(d_c - 2\,t_f) \times t_w$, (Wu et al 2005).

The experimental and analytical results of the steel beam connections to the reinforced concrete columns conducted by Sheikh (1987) indicated that the shear stress varied between $1.99\sqrt{f_c'}$ and $2.99\sqrt{f_c'}$ (in MPa), and the panel deforms as a monolithic unit. Thus, in Eq. (9) it is assumed that the entire concrete core area is effective in resisting the joint shear.

The shear strength at the web panel, V_{wfn} in Eq. (10) is provided by means of the shear yielding in the web, V_{wn} and the flexural rigidity of the flanges at the connection panel joint boundaries, V_{fn}.

Depending on the shear yield stress of $0.6F_{yw}$ of the Von Mises yield criteria, and as per the (AISC) LRFD specifications (1994), the web shear yield is calculated using Eq. (11), based on an average yield shear stress of $0.6F_{yw}$ acting over the horizontal web area within the joint panel.

The shear resistance of the flanges is calculated using Eq. (12), (Sheikh 1987 and Deirlein 1988). The web panel shear strength is given by the following equations:

$$V_{wfn} = V_{wn} + V_{fn} \tag{10}$$

$$V_{wn} = 0.6\, F_{yw}\, b_j\, t_w \tag{11}$$

$$V_{fn} = \frac{4\, M_{pf}}{d_b} \tag{12}$$

$$M_{pf} = \frac{F_{yf}\, t_f^2\, b_f}{4.0} \tag{13}$$

where F_{yw} and F_{yf} are the yield stress of the beam web and flange respectively, and t_w and t_f are the beam web and flange thicknesses, while b_j and d_b are the width of the panel joint and the depth of the beam respectively.

Figure 13 shows the hysteretic curves of shear deformation at the panel zone for both specimens. The shear force at the panel zone during testing is less than the calculated nominal shear capacity, due to the beam failure mechanism. Also, no inelastic deformations were observed, indicating that these equations used for calculating the nominal shear capacity for panel zone are conservative.

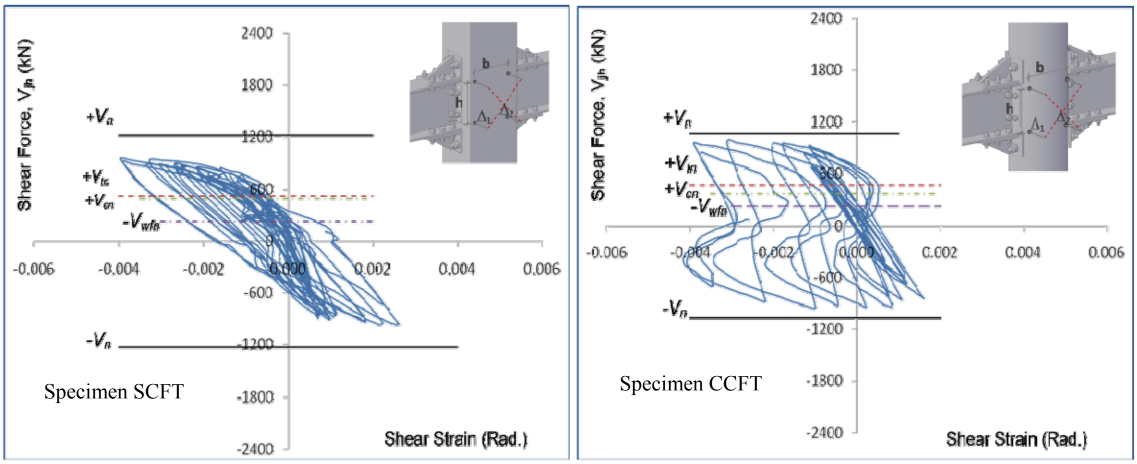


Figure 13. Hysteretic curves of shear deformation at the panel zone for both specimens

7 CONCLUSIONS

Previous research studies have indicated that the through beam type connection detail is an ideal rigid connection for attaching steel beams to CFT columns but this requires site welding. In this study a new connection is proposed where site welding can be avoided for through beam type connections for both circular and square CFT columns and the location of the yielding can be moved away from the column face. The results showed the capability of the proposed bolted bracket to develop the full plastic flexural capacity of the beam when the strong column-weak beam criterion is followed and the beams had inelastic rotational angles of 0.077 and 0.06 radians for the connection with square and circular CFT columns respectively, at their end, which were in excess of 0.04 radians as recommended by AISC (2002) for high seismic areas. The proposed, through beam type connection can be used in high seismic regions. Also the simulation results match well with the test results, and this demonstrates the ability of model developed using RUAUMOKO-2D software to simulate the cyclic behaviour of the through beam connection, very well.

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