CHAPTER 4

EXPERIMENTAL PROGRAM

4.1 GENERAL

In this study four half-scale interior connections with steel beams and concrete filled tubular (CFT) columns were tested under cyclic displacement controlled loading. Square and circular steel tubular columns were considered, with two different types of connections (end-plate type connections and through beam type connections). The purpose of the cyclic tests was to investigate the strength and inelastic rotation capacity of the connection subassemblies, and to determine their suitability of use in seismic force resisting moment frames. The fabrication and the testing of all the specimens were performed in the Structural Dynamic Laboratory of Anna University, Chennai, India.

The details of the testing program such as description of the test specimens, test setup, testing procedure, and test results are presented in this chapter.

4.2 TESTING PROGRAM

4.2.1 Design of the Test Specimens

The current design methodology adopted by the AISC seismic provisions 2011 requires that the specified inter-storey drift of a steel moment frame is accommodated through a combination of elastic and inelastic frame
deformations. In multi-storey frames, the greatest amount of energy dissipation capacity occurs when plastic hinges develop in the beams. This is due to the fact that collapse mechanisms comprised of beam hinges require more elements than mechanisms composed of column hinges. Consequently, the most desirable approach is a strong column, strong connection, and weak beam design philosophy. All specimens tested during this study were designed following this design philosophy.

**4.2.1.1 Design moment**

Inelastic behaviour in steel special moment frame structures is intended to be accommodated through the formation of plastic hinges at beam-column joints and column bases. Plastic hinges form through flexural yielding of beams and columns and shear yielding of panel zones, (Sumner 2003). It is expected that beams will undergo large inelastic rotations at targeted plastic hinge locations. The location of the formation of plastic hinges within the connecting beams is dependent upon the type of beam-to-column connections used. The expected locations of the plastic hinges within the frame (Figure 4.1) should be used to properly model the frame behaviour, and to determine the strength demands at the critical sections within the connections.

To satisfy the strong column-weak beam philosophy, a column-to-beam flexural strength ratio higher than 1.0 recommended by AISC (2002) can be adopted. Using the expected moment at the plastic hinge, $M_{pr}$, and the location of the hinge, the flexural strength demands at each critical section can be determined.
4.2.1.2 Beam design

Tests conducted as part of this study are intended to prequalify the two types of connections for use in seismic region. Universal Beam sections (UB) were used in this study and it was expected that beams would undergo large inelastic rotations at targeted plastic hinge locations, which might be at the ends of beams. According to AISC seismic provisions (2011), the expected yield stress \( (R_f, F_y) \) were calculated for the selected beam sections and were used in calculating the expected beam plastic strength, \( (M_{pl}) \) using Eq. (3.1), as described in Section 3.3.1.1. The dimensions and grade of steel used for the column sections are presented in Table 4.1.
Table 4.1 Dimensions and types of connection in the test specimens

<table>
<thead>
<tr>
<th>Designation</th>
<th>Column section (mm)</th>
<th>Material (MPa)</th>
<th>Steel beam section (mm)</th>
<th>Material (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specimen No.1</td>
<td>220×220×6 (Square)</td>
<td>Grade 300</td>
<td>UB 203×133×7.8×5.8</td>
<td>Grade 300</td>
</tr>
<tr>
<td>Specimen No.2</td>
<td>Ø =220; t=8 (Circular)</td>
<td>Grade 250</td>
<td>UB 203×133×7.8×5.8</td>
<td>Grade 300</td>
</tr>
<tr>
<td>Specimen No.3</td>
<td>220×220×6 (Square)</td>
<td>Grade 300</td>
<td>UB 203×100×9.4×6.0</td>
<td>Grade 250</td>
</tr>
<tr>
<td>Specimen No.4</td>
<td>Ø=220; t=8 (Circular)</td>
<td>Grade 250</td>
<td>UB 203×100×9.4×6.0</td>
<td>Grade 250</td>
</tr>
</tbody>
</table>
The width-thickness ratios b/t and D/t provided for both specimens were 34 and 27 respectively, satisfying the AISC 341 (2010) specification. Here D is the outer diameter of the circular CFT section, b is the inner width of the rectangular section, t is the thickness of CFT section, while E and $F_y$ are the modulus of the elasticity of the column and yield strength of the steel tube respectively. The moment capacity of the column was calculated based on the axial load applied on the column during the test using the analytical model developed by Elremaily and Azizinamini (2001). The squash load, $P_o$, was calculated as the summation of the ultimate axial capacities of both steel and concrete and is given by the following equation:

$$P_o = A_{st} F_y + A_c f'_c$$  \hspace{1cm} (4.1)

where, $A_{st}$ and $A_c$ is the area of the steel tube and the area of concrete filled tube respectively, $F_y$ is the tensile strength of the steel tube, $f'_c$ is the compressive strength of the concrete filled tube.

The columns of the test specimens were designed to remain elastic throughout the testing. The dimensions and grade of steel used for the beam sections are presented in Table 4.1.

4.2.2 Description of the Specimens

Four half-scale models of interior steel beam-to-CFT column subassemblies were tested. All the specimens were designed as per the AISC (2011). All connections, panel zones, and columns were designed to ensure that the connection would fully develop the strength of the beam, resulting in a ductile beam failure mode, satisfying the strong column-weak beam concept. As per the strength estimation, due to over strength beam action, the rods, panel zone, columns, and end-plate lift-off were expected to reach 94%, 65%, 68%, and 85% of their capacities respectively.
The details of the test specimens and the type of connections are given in Table 4.1. In Specimens No.1 and No.2, the beam-to-CFT column connections were extended end-plate moment connections, while in Specimens No.3 and No.4 the through beam type connections were used. It may be seen that the columns of Specimens No.1 and No.3 had square cross-sections, while Specimens No.2 and No.4 had circular cross-sections.

### 4.2.2.1 End-plate type connections

The test specimens with extended end-plate connections for square and circular CFT columns are shown in Figures 4.2a and b respectively.

![Test specimens of the end-plate type connections](image)

**Figure 4.2 Test specimens of the end-plate type connections**

**Specimen No.1**

The details of Specimen No.1 are shown in Figure 4.3. This specimen is composed of a concrete filled tubular column with a $220 \times 220$ mm square cross-section. Since the required dimensions are not available
commercially, the column was made by combining two equal L-shaped steel plates (Grade 300, $F_y$ (minimum)=300MPa) into a steel tube using full penetration butt welds with backing bars at the two corners. This column is connected to a universal steel beam UB 203×133×7.8×5.8 (Grade 300) with two flat stiffened extended end-plates, welded with fillet welds between the beam and the end-plate.

These end-plates were fastened to the square tube by 22mm diameter threaded rods passing through the column, as shown in Figure 4.4. There were four rods above and below the flange plate respectively, and in total eight rods were used in the east and west end-plates for fastening the end-plates with steel tube as shown in Figure 4.3.

Figure 4.3 Details of Specimen No.1
Figure 4.4 Rods of the flat end-plate type connection passing through the CFT column

Specimen No.2

The details of Specimen No.2 are illustrated in Figure 4.5. This specimen has a Grade 250 circular tube and two curved end-plates at the beam end. These curved plates (16mm thickness) were cut from a circular tube having its internal diameter equal to the external diameter of the CFT column.

Like Specimen No.1, in this specimen there were also four rods above and below the flange plate respectively, and in total eight rods were used in the east and west end-plates. The 22mm diameter rods were ordered and tested to ensure that $F_y$ (minimum) was 400MPa. The rods were threaded over 50mm from each end. These rods were passed through the column in “×” shape at the same level, as shown in Figure 4.6. To avoid conflict among the two rods, a special arrangement was made as explained below:

One of the rods was cut into two pieces, threaded, and then inserted into two nuts, which were welded to two flat plates (Figure 4.6a).
Figure 4.5 Details of Specimen No.2

(a) Arrangement of rods in “×”shape  (b) Rods through the circular column

Figure 4.6 Rods of the curved end-plate type connection passing through the CFT column

These plates facilitate room for insertion of the other rod. For the insertion of the rods in the column, the plate with the welded nuts was
inserted first from the top of the column, as it could not easily be passed through the wall like the other rod. Then the pieces of the rod were inserted from either side and fitted into the nuts. The second rod was passed straight through. An alternative connection with only straight rods and a sloping washer was not used here, but it would be expected to have more similarity to the rectangular arrangement.

A stiffener of a triangular plate of 140mm length, 75mm height and 12mm thickness in the plane of the girder web was provided on the outside of each flange, to stiffen the extended portions of the end-plate to reinforce the beam end, and to shift the plastic hinge location away from the end-plate weld, thus increasing the bending moment capacity of the beam end. Fillet welding of 6mm thickness was used here. All the welding work was done by a licensed fabricator.

At the end of the column in each specimen, steel square plates 6mm thick were welded, to support the wet concrete during the pour. A silicone layer, (as shown in Figure 4.7) was used to fill all the gaps from outside the column surface, to prevent leaking of the fresh concrete and water.

![Figure 4.7 Filling the voids using silicone](image)
The specimens were filled with concrete using a funnel. The concrete was filled up to the end of the column. A silicone layer was used to cover the top of the column after four hours of casting and therefore self-curing of concrete was ensured (Figures 4.8, 4.9 and 4.10).

Figure 4.8 Specimen after casting

Figure 4.9 Providing a silicone layer at the column top
When the strength of the concrete was developed after 28 days, the rods in the end-plate type connections were tightened, using a torque wrench (Figures 4.11 and 4.12), by applying a torque of 340Nm, resulting in a pretension force of approximately 77kN (55% of the proof strength level) in the rods.
4.2.2.2 Through beam type connections

The test specimens with through beam type connections for square and circular CFT columns are shown in Figures 4.13a and b respectively.

![Test specimens of through beam type connections](image)

(a) Specimen No.3 (Square CFT column)
(b) Specimen No.4 (Circular CFT column)
Specimens No.3 and No.4

The details of the through beam type connection for Specimens No.3 and No.4 are shown in Figures 4.14 and 4.15 respectively.

Figure 4.14 Details of Specimen No.3

These specimens consisted of steel beams passing through the column to represent an interior joint in a building. The specimens No.3 and No.4 had a square and a circular CFT column of Grade 300 and 250 respectively. The steel beam UB 203×100×9.4×6.0 of Grade 250 was used with less flange width, but having same weight, same cross sectional area and same moment of inertia, compared to that used in the end-plate type connection, to provide more space inside the column for pouring the concrete.

An opening in the shape of the steel beam but with 2mm oversize was cut in the steel tube (column), to allow the girder to pass through the column. It was cut with the gas torch. 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.

Figure 4.15 Details of Specimen No.4

The subassembly consisted of four bolted brackets between the beam and column (flat bolted brackets and curved bolted brackets) for Specimen No.3 and Specimen No.4 respectively, as shown in Figure 4.16. 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.

The disadvantage of providing the bolted bracket is that the holes need to be made in the beam flanges, which may depend on their size and the bolt pre-stress, resulting in decreased strength and deformation capacity.
These holes are predrilled on the top and bottom flange of the North and South beam 12.5mm in diameter, and positioned corresponding to those in the bolted bracket. This resulted in a steel ratio calculated as the net area to gross area \((A_{\text{net}}/A_g)\) of 0.84 for the section and 0.75 for the flange. Also the ratio \((A_g, \text{flange} \times F_{y, \text{flange}}) / (A_{\text{net}, \text{flange}} \times F_{u, \text{flange}})\) was 0.86. The brackets in Figure 4.16 were installed entirely by bolting to the column, using rods passing through the column. The bolts were snug tightened, as this provides low pre-stress and is likely to provide the most critical conservative condition.

![Figure 4.16 Bolted bracket in the through beam type connection](image)

4.2.3 Test Setup

The test setup shown in Figure 4.17 was designed to test the interior beam-to-column joint models simulating seismic loading conditions. Pseudo-controlled hydraulic actuators having a maximum force capacity of 490kN and a stroke of ±100mm were positioned vertically, as shown in Figure 4.17, to apply equal and opposite cyclic displacement controlled loading to the free ends of the steel beams to simulate the deformed shape of an interior joint in a building subjected to lateral loads. The bottom and the top ends of the columns were connected to the strong reaction floor and the reaction frame respectively, as shown in Figure 4.18.
Figure 4.17 Test setup

Figure 4.18 End supports of the column

(a) Bottom end of the column  (b) Top end of the column
Pin supports were provided between the actuators and the steel beam, to allow the beam end to move freely in the vertical direction, as shown in Figure 4.19.

**Figure 4.19 Pin support between the actuators and the steel beam**

The column was subjected to a constant axial load, equivalent to 11% of the axial strength capacity of the CFT column (based on the actual material strength and geometric properties of steel and concrete), to provide the necessary column confinement to the connection. A specimen with the testing arrangement is shown in Figure 4.20.

### 4.2.4 Instrumentation

Six laser sensors with a reading error of less than 8\(\mu\)m were used to measure the beam vertical displacements (Figure 4.21). They were fixed above the top flange of the steel beam.
Figure 4.20 Specimen with the testing arrangement

Figure 4.21 Fixing the laser sensor at the end of the beam
Linear strip type electrical resistance strain gauges (120Ω) of 20mm length were placed on the center (unthreaded region) of each rod to obtain the bolt forces. The strain gauge readings were calibrated by conducting tensile tests on the rods using universal testing machine.

The acquisition of the rod strain was useful for determining the bolt forces throughout the test, as well as for finding the pre-tensioning force in the bolts accurately.

For through beam type subassemblies, the embedded web strains were measured with gauges of 90mm length at 45°, one in front of, and the other behind the web, as shown in Figure 4.22.

![Figure 4.22 Fixing the strain gauge at 45° on the beam web](image)

The shear deformation of the panel zone was calculated, using small linear variable differential transducers (LVDTs) installed in “×” shape, as shown in Figure 4.23. Also, at the beam tip, a 490kN capacity tension-compression load cell was installed between the actuators and the beams, to measure the force induced at the load point.
4.2.5 Data Acquisition

A sixteen channel data acquisition system (DEWE-43) was used to monitor and control the displacement and force feedback signals. The test control and the data acquisition system were run by a Windows-based control and acquisition program called “DEWE Soft 7”. The data acquisition system recorded the data at a rate of 100 points per second (100 Hz). Figure 4.24 shows the data acquisition system used and Figure 4.25 shows the column shear force vs. drift ratio and load-displacement responses of Specimen No.1 recorded by the acquisition program.

4.2.6 Loading History

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 4.26.
Figure 4.24 Data acquisition system (DEWE-43)

Figure 4.25 Recording the data using data acquisition program “DEWE Soft 7”
Table 4.2 presents the details like the peak beam rotation at each load step, number of cycles in each load step and the test termination details for all the specimens.

4.2.7 Testing Procedure

The specimen was installed in the loading frame. The test setup, instruments and the loading history are explained in the previous sections. The instrumented rods were connected to the data acquisition system, and all the readings were set to zero. The rods of the end-plate type connections were tightened using a torque wrench. During tightening, the rod tension was monitored using the data acquisition system. Calibration values were checked and stored in the system. The test began with the recording of an initial zero-load reading. Axial loads were applied to the columns first. This value was maintained constant during the entire period of the test. Actuators were used to apply equal and opposite vertical cyclic displacements on the two ends of the steel beams. The specimens were loaded according to the loading history

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Figure 4.26 Cyclic loading history
as presented in Table 4.2. The loading history prescribes a series of load steps and the number of cycles for each step, as shown in Figure 4.26. The load steps were continued until failure of the specimens occurred.

### Table 4.2 Loading history

<table>
<thead>
<tr>
<th>Load step No.</th>
<th>Peak beam rotation (Radian)</th>
<th>Number of cycles</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.0086</td>
<td>3</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>0.0150</td>
<td>3</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>0.0322</td>
<td>3</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>0.0483</td>
<td>3</td>
<td>-</td>
</tr>
<tr>
<td>5</td>
<td>0.0540</td>
<td>2</td>
<td>For Specimens No.1 and No.2 tests terminated after the load step 5</td>
</tr>
<tr>
<td>6</td>
<td>0.0591</td>
<td>3</td>
<td>For Specimen No.4 test terminated after the load step 6</td>
</tr>
<tr>
<td>7</td>
<td>0.066</td>
<td>3</td>
<td>-</td>
</tr>
<tr>
<td>8</td>
<td>0.0770</td>
<td>2</td>
<td>For Specimen No.3 test terminated after the load step 8</td>
</tr>
</tbody>
</table>

### 4.3 MATERIAL TESTS

#### 4.3.1 Tensile Coupon Testing

Materials for the tensile test coupons were cut from the flanges and webs of the steel beams, circular tube, square tube, flat and curved end-plates, as shown in Figure 4.27. Procedures as per IS 1608 (2005) were followed to test the samples. Tensile tests were conducted on the samples collected for the coupon test using a 2500kN capacity universal testing machine (Figure 4.28). The results of these tests are given in Table 4.3.
Figure 4.27 Control samples for the coupon test

Figure 4.28 Universal testing machine used for the tensile test
Table 4.3 Material strengths of the steel samples

<table>
<thead>
<tr>
<th>No.</th>
<th>Sample</th>
<th>Elongation (%)</th>
<th>Yield strength (MPa)</th>
<th>Ultimate strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Beam section UB 203×100 (Grade 250)</td>
<td></td>
<td>38.0</td>
<td>293.80</td>
</tr>
<tr>
<td>2</td>
<td>Web</td>
<td></td>
<td>39.4</td>
<td>264.20</td>
</tr>
<tr>
<td>3</td>
<td>Beam section UB 203×133 (Grade 300)</td>
<td></td>
<td>37.2</td>
<td>342.14</td>
</tr>
<tr>
<td>4</td>
<td>Web</td>
<td></td>
<td>19.2</td>
<td>415.44</td>
</tr>
<tr>
<td>5</td>
<td>Square tube (Grade 300)</td>
<td></td>
<td>36.0</td>
<td>326.27</td>
</tr>
<tr>
<td>6</td>
<td>Circular tube (Grade 250)</td>
<td></td>
<td>21.3</td>
<td>285.91</td>
</tr>
<tr>
<td>7</td>
<td>Flat end-plate (Grade 250)</td>
<td></td>
<td>29.8</td>
<td>297.58</td>
</tr>
<tr>
<td>8</td>
<td>Curved end-plate (Grade 300)</td>
<td></td>
<td>30.0</td>
<td>362.50</td>
</tr>
<tr>
<td>9</td>
<td>Rod Ø22</td>
<td></td>
<td>10.9</td>
<td>406.62</td>
</tr>
</tbody>
</table>

4.3.2 Concrete Compressive Strength

Concreting was carried out in the laboratory according to ACI 301 (2002). The concrete consisted of crushed stone processed from natural rock in accordance with ASTM C33 (1989) specifications, with maximum size of aggregate 10mm and a slump value of 120mm. The design compressive strength of concrete was 30MPa. The concrete cube strength $f_{cu}$ was obtained by testing 150mm concrete cubes. The cylinder strength $f'_c$ was obtained by testing 300mm×150mm concrete cylinders. Testing of the control samples were carried out following the guidelines of the testing requirements as per ASTM C31 and C39 (1989).
Figure 4.29 shows the control samples of the concrete. These samples were tested, using the compression testing machine as shown in Figure 4.30. The values of the concrete strength at the age of 28 days, and on the day of testing, are summarized in Table 4.4.

![Figure 4.29 Control samples of the concrete after casting](image)

![Figure 4.30 Compression testing machine](image)
**Table 4.4 Concrete compressive strength**

<table>
<thead>
<tr>
<th>Description</th>
<th>Specimen No.1</th>
<th>Specimen No.2</th>
<th>Specimen No.3</th>
<th>Specimen No.4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strength at 28 days (MPa)</td>
<td>37.2</td>
<td>47.7</td>
<td>33.4</td>
<td>40.9</td>
</tr>
<tr>
<td>Compressive strength on the day of test (MPa)</td>
<td>39.8</td>
<td>49.7</td>
<td>37.7</td>
<td>44.8</td>
</tr>
</tbody>
</table>

**4.4 EXPERIMENTAL RESULTS**

The parameters used to evaluate the specimen performance, were the maximum applied moment and the maximum rotation angle, with the corresponding moment. The applied moment was calculated by multiplying the applied load by the distance from the load application point to the face of the column. The plastic flexural strengths of the beams for all the specimens were calculated, based on the material strengths measured in the coupon test.

For the tested interior sub assemblages, the applied beam rotations were calculated by dividing the beam tip displacement by the distance from the beam tip to the column face using laser sensor readings taken at the end of the beam.

Another important parameter used to evaluate the specimen performance is the column shear force versus drift ratio. For each sub-
assemblage, it was assumed that the maximum dependable flexural strengths of the beam sections at both sides of the column had developed simultaneously while undergoing equal rotations. As shown in Figure 4.31, the CFT column deformed under the action of storey shear \((V_C)\) which is calculated using Eq. (4.2):

\[
V_C = \left[ P_1 + P_2 \right] \frac{L}{2h_C}
\]

(4.2)

where \(V_C\) is the horizontal storey shear force. \(P_1\) and \(P_2\) are the beam end forces, while \(L\) and \(h_C\) are the beam pin to pin span length and the column height respectively.

![Figure 4.31 Determination of inter-storey displacement and forces](image)

Since the CFT column was not designed to develop plastic hinges, it was assumed to remain elastic throughout the test. The column displacement was estimated by applying the geometrical relationship which is as follows:
\[
\tan \theta_{\text{col}} \approx \frac{\Delta_c}{h_c} \tag{4.3}
\]

\[
\tan \theta_N \approx \frac{\Delta_N}{(L/2)} \tag{4.4}
\]

\[
\tan \theta_S \approx \frac{\Delta_S}{(L/2)} \tag{4.5}
\]

Substituting and getting $\Delta_C$ as:

\[
\Delta_C = [\Delta_N + \Delta_S] \left[ \frac{h_c}{L} \right] \tag{4.6}
\]

In the current study, the storey drift ratio was calculated based on Eq. (4.7):

\[
\text{Drift (\%)} = \frac{\Delta_c}{h_c} \times 100\% \tag{4.7}
\]

The results obtained for all the four specimens are summarized in Table 4.5.

4.4.1 General Behaviour

All 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. No inelastic deformation was found at the end-plates throughout the loading, while the column and panel zone remained elastic during all stages of loading.

It was found that the total beam rotation for all the specimens was greater than 0.04 radians, satisfying the recommendation given by AISC (2002) for a composite special moment resisting frame.
### Table 4.5 Summary of the test results

<table>
<thead>
<tr>
<th>Description</th>
<th>Plastic moment $M_p$ (kNm)</th>
<th>Max. test flexural moment (kNm)</th>
<th>The ratio of $M_{\text{test}} / M_p$</th>
<th>Ult. Rotation angle, $\theta_u$</th>
<th>Max. shear $V_C$ (kN)</th>
<th>Column Drift (%)</th>
<th>Inelastic deformation mode</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$+M_{\text{test}}$</td>
<td>$-M_{\text{test}}$</td>
<td>$+\theta_u$</td>
<td>$+V_C$</td>
<td>$-V_C$</td>
<td></td>
</tr>
<tr>
<td>Specimen No.1</td>
<td>95.50</td>
<td>126.10</td>
<td>118.96</td>
<td>1.32</td>
<td>1.25</td>
<td>0.054</td>
<td>5.4</td>
</tr>
<tr>
<td>Specimen No.2</td>
<td>95.50</td>
<td>127.68</td>
<td>131.29</td>
<td>1.34</td>
<td>1.37</td>
<td>0.054</td>
<td>5.4</td>
</tr>
<tr>
<td>Specimen No.3</td>
<td>75.26</td>
<td>114.28</td>
<td>115.79</td>
<td>1.52</td>
<td>1.54</td>
<td>0.077</td>
<td>7.7</td>
</tr>
<tr>
<td>Specimen No.4</td>
<td>75.26</td>
<td>115.09</td>
<td>108.40</td>
<td>1.53</td>
<td>1.44</td>
<td>0.0591</td>
<td>5.9</td>
</tr>
</tbody>
</table>
4.4.2 Test Observations

4.4.2.1 Specimen No.1 (Square CFT column with flat extended end-plate connection)

This specimen performed in a ductile manner with a plastic hinge at the beam end near the column face. Local buckling took place mainly behind the stiffener, away from the end-plate. Based on visual observation, the local buckling at the bottom flange was observed during the first cycle of the fourth loading step (beam rotation of 0.048 radians), as shown in Figure 4.32. By the end of the third cycle at this loading step, a small gap between the end-plate and column was observed. The test was stopped at the end of the fifth loading step (with beam rotation of 0.054 radians), due to a fracture in the welding of the end-plate as shown in Figures 4.33 and 4.34. The small gap behind the end-plate occurred at a force level consistent with that predicted for the rod pre-tensioning used, as discussed previously (Figure 4.34). If pre-tensioning had been conducted to the full rod proof load, it is expected that no separation would have occurred.

Figure 4.32 Specimen No.1 at the beam rotation of 0.048 radians
Figure 4.33 Failure condition of Specimen No.1

Flange buckled in & web buckled out

Flange buckled out & web buckled in

3mm Gap

Figure 4.34 Fracture in the welding of the end-plate in Specimen No.1

Fracture in welding then propagate in the beam web
4.4.2.2 Specimen No.2 (Circular CFT column with curved extended end-plate connection)

This specimen performed in a ductile way similar to Specimen No.1, with a plastic hinge at the beam end near the column face. The top and bottom flange started to bend at the first cycle of the fourth loading step (beam rotation of 0.048 radians); then, a small gap between the end-plate and column was observed in the third cycle of the same loading step. Local buckling of the beams was observed as shown in Figures 4.35 and 4.36. The beam flanges buckled in and the web plate buckled out. The test was stopped at the end of the fifth loading step (with beam rotation of 0.054 radians), due to a fracture in the welding between the beam end and the curved end-plate.

![Figure 4.35 Failure condition of Specimen No.2 at the end of the test](image)

Flange buckled in & web buckled out
Flange buckled in & web buckled out
4.4.2.3 Specimen No.3 (Square CFT column with through beam connection)

This specimen performed in a ductile manner with the development of plastic hinges in the beam. Local buckling at the bottom flange was first observed during the third cycle of the fourth loading step (with beam rotation of 0.048 radians); the beam web consequently yielded at the first cycle of the fifth loading step (with beam rotation of 0.054 radians), as shown in Figure 4.37. At a rotational angle of 0.066, in the seventh step of loading, a small gap between the bolted bracket and the column was observed, and significant yielding occurred in both the beams as shown in Figure 4.38.

Figure 4.37 Specimen No.3 at the beam rotation of 0.048 radians
Figure 4.38  Tearing of the beam at the end of the bolted bracket at the rotational angle of 0.066 radians

The test was ended at the second cycle of the eighth loading step (with beam rotation of 0.077 radians), owing to the significant strength drop caused by the tearing of the beam at the end of the bolted bracket through the line of bolts away from the column, as shown in Figure 4.39. The tube wall (column) showed no apparent signs of local distress.

Figure 4.39  Failure condition of Specimen No.3 at the end of the test
4.4.2.4 Specimen No.4 (Circular CFT column with through beam connection)

This specimen performed in a way similar to Specimen No.3. Bending in the bottom flange was noticed at the fifth loading step (with beam rotation of 0.054 radians) as shown in Figure 4.40.

![Start of the bending of bottom flange](image)

**Figure 4.40 Specimen No.4 at the beam rotation of 0.054 radians**

Tearing of the top flange of the beam started in the same loading step. The test was terminated at the first cycle of the sixth loading step (with beam rotation of 0.059 radians), due to severe tearing in the top flange of the beam, at the end of the connection away from the column, and eventually the tearing of the plate propagated into the web, as shown in Figures 4.41 and 4.42. Also, the tube wall showed no apparent signs of local distress.
Figure 4.41  Tearing of flanges and web of the beam at the end of the bolted bracket at the rotational angle of 0.059

Figure 4.42 Failure condition of Specimen No.4 at the end of the test
4.4.3 **Hysteretic Curves of Column Shear versus Drift Ratio**

The hysteretic curves of the column shear versus drift ratio for both types of connections (end-plate type and through beam type) are shown in Figures 4.43 and 4.44 respectively. The column shear force and the storey drift ratio are calculated based on Eqs. (4.2) and (4.7) respectively, as described in Section 4.4.

The column shear force versus drift ratio for Specimen No.1 is shown in Figure 4.43a. The figure shows a stable behaviour with maximum drift ratio of 5.4% and the maximum column shear forces were 172.98kN and 179.13kN in the positive and negative loading directions respectively.

The column shear force versus drift ratio for Specimen No.2 is shown in Figure 4.43b. The maximum column shear forces were 175.77kN and 181.57kN in the positive and negative loading directions respectively. The figure illustrates, that the overall behaviour of Specimen No.2 is similar to that of Specimen No.1 with slight increase in the maximum values of column demand. The ultimate drift ratio for both the specimens was 5.4%.

The column shear force versus drift ratio for Specimen No.3 is shown in Figure 4.44a. The maximum column shear forces were 158.02kN and 161.18kN in the positive and negative loading directions respectively. The figure shows that the Specimen No.3 sustained a drift ratio upto 7.7 % before strength deterioration was apparent. Further, the hysteretic loops were full and stable even at larger drift levels.

The column shear force versus drift ratio for Specimen No.4 is shown in Figure 4.44b. The maximum column shear forces were 162.03kN and 157.72kN in the positive and negative loading directions respectively. The figure shows stable hysteretic loops and Specimen No.4 sustained a drift ratio of 5.9%.
Figure 4.43  Column shear force vs. drift ratio for the end-plate type connections
Figure 4.44  Column shear force vs. drift ratio for the through beam type connections
4.4.4 Hysteretic Curves of Beam Moment versus Total Rotation

The total beam rotation was calculated using the laser sensor readings as the relative displacement between the beam end and the column face whereas the local rotations of the plastic hinges were found from the displacement readings taken on either side of the plastic hinge. The beam moment was calculated as discussed in Section 4.4.

Figures 4.45 and 4.46 show the beam moment-rotation (total and local) relations respectively, for the Specimen No.1 with flat end-plate connection. The connection developed approximately 1.3 and 1.25 times the plastic flexural strength of the beam in the positive and negative loading directions respectively. The subassembly sustained a rotational angle of 0.054 radians.

Figures 4.47 and 4.48 demonstrate stable hysteretic behaviour of the beam moment-rotation (total and local) relations respectively, for the Specimen No.2 with curved end-plate connection. It can be seen from this figure that the curved end-plate connection developed approximately 1.34 and 1.37 times the plastic flexural strength of the beam in the positive and negative loading directions respectively. It also showed similar performance to that of flat end-plate connection and exhibited good ductility.

For through beam type connections hysteretic behaviour of the beam moment-rotation (total and local) for Specimen No.3 are shown in Figures 4.49 and 4.50 respectively. It can be clearly seen that this connection developed approximately 1.52 and 1.54 times the plastic flexural strength of the beam in the positive and negative loading directions respectively. This specimen exhibited stable behaviour at inelastic rotations upto 0.077 radians. Hence, the through beam type connection can be used in high seismic regions.
For Specimen No.4 with circular CFT column, the hysteretic curves of beam moment versus total rotation and beam moment versus local rotation are shown in Figures 4.51 and 4.52 respectively. It can be noticed that the flexural strength of this connection in the positive and negative loading directions were 1.53 and 1.44 times the plastic flexural strength of the beam respectively. This specimen exhibited ductile behaviour and stable inelastic rotation of 0.059 radians.

It can be concluded from the hysteretic curves of beam moment vs. total rotation for the end-plate type connections and through beam type connections that the test specimens have shown good ductile response, and both the types of connections demonstrated stable hysteretic behaviour.

**4.4.5 Envelope Curves**

Figures 4.53a and b show envelope curves of the beam moment versus rotation and column shear force versus displacement for all the four specimens. At the early stage of the testing (upto rotational angle of 0.01radians) all tested specimens showed approximately similar beam strength (with respect to beam rotation). During large rotation levels of above 0.01radians, the rate of increase in beam strength became significantly higher in both directions of loading for the end plate type connections, similar performance were noticed in the column shear force versus displacement demands.

Finally, it can be concluded that the two types of connections reveal a similar performance. The performance of a through beam type connection was better, especially in terms of ductility, with a slight reduction in the strength resistance capacity.
Figure 4.45 Beam moment vs. total rotation for Specimen No.1

Figure 4.46 Beam moment vs. local rotation for Specimen No.1
Figure 4.47 Beam moment vs. total rotation for Specimen No.2

Figure 4.48 Beam moment vs. local rotation for Specimen No.2
Figure 4.49 Beam moment vs. total rotation for Specimen No.3

Figure 4.50 Beam moment vs. local rotation for Specimen No.3
Figure 4.51 Beam moment vs. total rotation for Specimen No.4

Figure 4.52 Beam moment vs. local rotation for Specimen No.4
Figure 4.53 Envelope curves for beams and columns of all the tested specimens
4.4.6 Energy Dissipation Capacity

Energy dissipation capacity is an indication of the ability of a structure to resist earthquakes, as it contains aspects of both strength and deformation capacity. The dissipated energy is defined as the area enclosed by the hysteretic force-displacement curve. As all the four specimens had a strong panel zone and column, energy was dissipated through beam flexural yielding and subsequent buckling, until the strength was reduced to less than 70% of its peak value. Sometimes fracture was observed in the final stage of the strength loss. Figure 4.54 shows the cumulative dissipated energy for all the four specimens.

![Figure 4.54 Energy dissipation capacity for all specimens](image)

The cumulative energy dissipation capacity is relatively the same for the Specimens No.1, 2 and 4 (roughly 70-95kNm). However, it was likely that the Specimen No.3 eventually underwent larger energy dissipation capacity than the other specimens, because it sustained a greater number of cycles. The dissipation of energy in the through beam type connections was due to the yielding of the beam flange. At rotations between 0.02 and 0.05, the stronger specimens had greater energy dissipation capacity. It is clear that
the specimens with greater deformation capacity had greater energy dissipation capacity at the end of the test.

4.4.7 Strains in the Rods

All the rods in the extended end-plate subassemblies were initially strained by using a hand torque wrench. Figures 4.55 and 4.56 show the strains in different rods in Specimen No.1. The location of the strain gauges in the rods are clearly shown in the figures.

The hysteretic loops of the rods in the first and second layers (top rods) are stable (Figures 4.55a and b), while in the third and fourth layers (bottom rods) the curves are unstable (Figures 4.56a and b) due to the occurrence of a sudden release in the rod strain at the fifth loading step (with beam rotation of 0.054 radians), because of the welding fracture between the bottom flange of the beam and the end-plate, as observed at end of the test.

The rods in the different layers (Figures 4.57a and b for the first and second layers (top rods) and Figures 4.58 a and b for the third and fourth layers (bottom rods) respectively) in Specimen No.2 performed in a way similar to those in Specimen No.1, with slightly lower strains. This specimen had crossing rods and curved end-plates. Any outward movement of the end plate due to lift off, would therefore induce not only axial force, but also bending in the rods.

For the through beam type connections, the strains in the web/rods for Specimens No.3 and No.4 are shown in Figures 4.59 to 4.62. Embedded web strain gauges were placed diagonally on the front and back of the panel zone with additional strain gauges fixed on the rods passing through the depth of the column, in the top and bottom layers in both the Specimens No.3 and 4. For Specimen No.3 it can be noted that the strain in the web of the steel beam within the panel zone (Figures 4.59a and b) was always less than 0.001 μ
mm/mm; this means that the web behaves elastically in this region. The strains in the top rods of Specimen No.3 (Figure 4.60a) exhibited relatively stable hysteretic loops with no pinching at the early stages of the test (rotational angle less than 0.01 radians). However, the rate of increase in the strain became higher after that, and the maximum value of the strain was greater than 6500 μ mm/mm at a rotational angle of 0.048 radians. The strains in the bottom rods of the Specimen No.3 (Figure 4.60b), were relatively similar to those in top rods at the early stages of the test, and the yielding was noted at a rotational angle of 0.015 radians. After that, continuous increases in the strain values were noted.

In Specimen No.4, the embedded strains in the front and back web of the beam (Figure 4.61a and b) behaved elastically at the early stages of the test, while at the later stages of the test the strains were relatively higher, (greater than 4000 μ mm/mm) for both faces. The strains in the top and bottom rods which were placed in “×” shape of Specimen No.4 (Figures 4.62a and b), exhibited lesser strain compared to that of Specimen No.3, even at later stages of the test and the maximam value was less than 2000 μ mm/mm. It was found that the specimen which had crossing rods was subjected to relatively lesser axial forces compared to those of Specimen No.3.

4.5 SUMMARY

In this research, experiments were conducted to investigate the behaviour of two different types of interior connections, between the steel beam and the square or circular CFT columns. The two connections considered were (i) End-plate type connections, where shop welded, flat and curved extended end-plates were bolted to the CFT column with steel rods passing through the column, and (ii) Through beam type connections, where the beam passes through the joint, and is connected with additional bolted brackets. The experiments demonstrated that:
Figure 4.55 Strains in the top rods of Specimen No.1
Figure 4.56 Strains in the bottom rods of Specimen No.1
Figure 4.57 Strains in the top rods of Specimen No.2
Figure 4.58 Strains in the bottom rods of Specimen No.2
Figure 4.59 Strains in the beam web of Specimen No.3

(a) Strain in front of the beam web of Specimen No.3

(b) Strain in back of the beam web of Specimen No.3

Figure 4.59 Strains in the beam web of Specimen No.3
Figure 4.60 Strains in the rods of Specimen No.3

(a) Strain in the top rods of Specimen No.3

(b) Strains in the bottom rods of Specimen No.3

Figure 4.60 Strains in the rods of Specimen No.3
(a) Strain in front of the beam web of Specimen No.4

(b) Strain in back of the beam web of Specimen No.4

Figure 4.61 Strains in the beam web of Specimen No.4
Figure 4.62 Strains in the rods of Specimen No.4

(a) Strain in the top rods of Specimen No.4

(b) Strain in the bottom rods of Specimen No.4
The flat and curved extended end-plate connections to the rectangular and circular CFT columns respectively, using rods passing through the composite column, reached the drift ratio of more than 5%, due to formation of plastic hinges in the beams away from the beam/column interfaces. The rods provided a direct load path through the column, causing compression only on the outside of the column. This resulted in a stiff load path, and no damage was observed in the column tube. End-plate type connections for both circular and square CFT columns showed similar performance.

The through beam type connection with no welding and bolted steel bracket to the beam and column, also showed stable behaviour to drift ratio greater than 5% (should be > 4% as per in AISC (2002)). In these tests the ratio \( \frac{A_g \times F_y}{A_{net} \times F_u} \) for the flange was 0.86, due to holes in the beam flanges. However, if this ratio were less than this, it is possible that premature fracture through the holes could significantly reduce the drifts obtained.

The cumulative energy dissipation capacity of the Specimens No.1, No.2 and No.4 were 71.6, 79.2, and 92.1 kNm respectively, roughly in the range of 70-95 kNm. For Specimen No.3, the cumulative energy dissipation capacity was 156.0 kNm, which is 117% higher than Specimens No.1 because it sustained more number of cycles.

The through beam type connection was found to perform better compared to the end-plate type connection in terms of energy dissipation capacity.

In terms of the flexural moment capacity, the end-plate type connection was found to have slightly higher (8-12%) values compared to that of through beam type connection.