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Comparative Study on Hysteretic Performance of Semi-Through Connections in CFT Beam - Column Joint

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Abstract: The concrete filled tubes (CFT) have the complete utilization of strength of its components both steel and concrete due to its peculiar positioning. The presence of outer steel improves the confinement of concrete which leads to high stiffness and strength, where as core concrete will support the steel tube from local buckling and enhance the overall performance. The application of CFT is limited due to the lack of design guidelines for the joint between CFT and structural steel beams especially in seismic regions. This paper attempts to evaluate and compare the hysteretic performance of two extended end plate exterior beam column connection suitable for square and circular CFT by investigating the strength, stiffness, ductility and energy dissipation. The results indicate the adaptability of both connections for Special moment resisting structure with consistently stable, ductile and good energy dissipation nature.

Index Terms: CFT column, Cyclic loading, Through bolt.

I. INTRODUCTION

Recent research on CFTs focuses on the methods to improve the strength of joints to avoid connection failure. Strength hierarchy is maintained in such a way that under seismic action the ductility of components is fully utilized well before the failure of the connection. During a seismic event the overall performance of the CFT building will be characterized by the connection pattern. The connection between steel beams and CFT columns can be broadly classified into two categories namely exterior and interior connections. Exterior connections are made by attaching the steel beam to the outer steel tube alone, which may lead to high distortion on tube wall. From the literature review joint failures reported in Northridge Earthquake were weld failures, which are categorized as exterior connections. The second type is interior connection which allows stress transfer to concrete core also. This is accomplished by penetrating the extended beam elements or the beam itself through the column. Even though this type of connection will improve the seismic performance, some practical construction difficulties prevail. Alternate solution to these problems is use of semi through type connections in which the entire beam is not passing through, instead extended anchor rods or bolts are transmitting the stresses to concrete core and thus making the confined concrete to actively participate in lateral stability. The selection of semi through connection has to be properly analyzed for easiness in construction, location of plastic hinge and cost benefit ratio[1-5].

Circular and square CFT members are commonly used in construction. In comparison, circular CFT has better performance than square CFT due to higher confinement. But the application of circular CFT is limited due to the difficulties in joint configuration and performance [6-8]. Current study aims at the performance evaluation of semi rigid semi through connection of a circular CFT using curved extended end plate and that of a square CFT using ordinary flat end plate under cyclic loading. This kind of bolted connection helps to avoid field weld completely which in turn increases the speed as well as quality of construction [9]. This paper describes the guidelines for design of CFT connections and methodology for testing. The performance of exterior joint connection under cyclic loading were analysed and results were compared. The test results also shows the adaptability of split bolt assembly for a circular CFT connection, as its performance is comparable with the performance of a square CFT with normal through bolt assembly[10].

II. EXPERIMENTAL PROGRAMME

A. Specimen Design

The design criteria selected was Four Bolt Stiffened Extended (4ES) End-Plate connection configuration, as per ANSI/AISC 358-10 capacity design approach [11]. The presence of stiffener helps to push the location of plastic hinge away from the face of column which will in turn improve the joint performance [12]. End plate thickness was arrived on the assumption to avoid prying force, which leads to the fixation of required diameter of bolts and pretension force. Representative sub assemblage of exterior joints were selected by assuming inflection points on the midspan of beam and column.

Square CFT joint was considered as control specimen and its modified version was adopted for circular CFT specimen.

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B. Test Specimen Fabrication

CFT column cross sectional were fixed by satisfying the width thickness ratio criteria on AISC 341(10) and also the section availability in market. Throughout the test the column should remain elastic this criteria was used for the selection of beam. Details of the specimens are shown in Table I.

Table I. .Details of Specimens

Specimen Designation	Particulars	Dimension
1ESFF220	Exterior joint with square CFT using full bolt assembly with flat end plate	Column 220×220×8mm Beam ISMB 175
2ECSC220	Exterior joint with circular CFT using split bolt assembly with curved end plate	Column 220mm dia 8mm thick Beam ISMB 175

Control specimen 1ESFF220 illustrated in Fig.1(a) composed of square tube 220×220×8mm with 300 Grade and beam ISMB175 with 250 grade. The steel beam is welded to extended end plate of 16mm thick using 6mm fillet weld at factory. 10mm thick triangular Stiffeners were provided above and below the top and bottom flanges. 16mm rod with 8.8 grade were specially fabricated to form through bolts.

Specimen 2ECSC220 shown in Fig. 1(b) was fabricated with circular tube of 220 mm diameter and 8mm thick with 300 Grade and beam remains same. For the connection curved endplate was chosen with a special split bolt assembly which allows diametrical intersection of through bolt at same level as shown in Fig. 2.

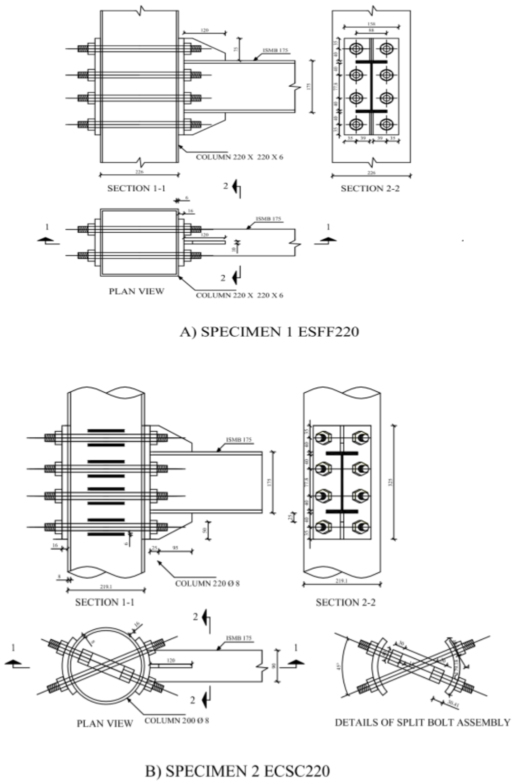

Fig.1. Exterior Joint Details

M30 grade concrete from 12mm aggregate, with plasticizer for 100mm slump was used to fill the steel tube. Mix design was carried out as per IS10262-2009 by considering good workability without compromising the strength requirement. Overall height of 1500mm and cantilever projection of 800mm were selected for all specimens considering the convenience of testing. After the fabrication of assembly the through bolts were placed as snug tight. Bolts were post tensioned after 28 days of concrete placing inside the tube. 100Nm torque was applied on either side of the rod using a calibrated torque wrench.

Mechanical properties of steel components used for CFT column, beam and HSFG bolt were determined using tension test and the values are presented in Table II, which shows that all the test results satisfies the design requirement. Concrete strength properties were characterized by cube and cylinder compression test and tensile properties using split tensile test.

Table II. . Material Properties of Components

Components	Yield Strength (MPa)	Ultimate Strength (MPa)
HSFG bolt (8.8 grade)	640	835
Steel Tube square	322	465
Steel Tube Circular	346	492
ISMB175 (flange)	284	393
(web)	268	381

Fig.2. Split Bolt Assembly

C. Test setup and Instrumentation

The test setup shown in Fig 3 was designed to simulate seismic loading effect on exterior joint by giving displacement controlled cyclic loading vertically on the free end of the beam by the use of manually controlled 750kN hydraulic actuator. 15% of the axial capacity of the column was given as static seating load over the top of the column and this minimum load was maintained throughout the test. Bolt prestress level also kept same for both specimen and its variation was monitored during the testing. High precision LVDT of LC 0.001 were used to capture the displacements accurately. Universal load cells were placed above the double acting hydraulic jack to measure both push as well as pull over the cycles. 5mm electric foil strain gauges were attached over the bolts and other critical locations of the specimen were high stress variations are expected. All data were collected by a 40 channel data acquisition system and recorded in an automated computer using lab view software with 80Hz frequency.

The real time response was monitored continuously and the feedback used to control displacement excitation of quasi static cyclic loading. Experimental setup photograph is shown in Fig. 4.

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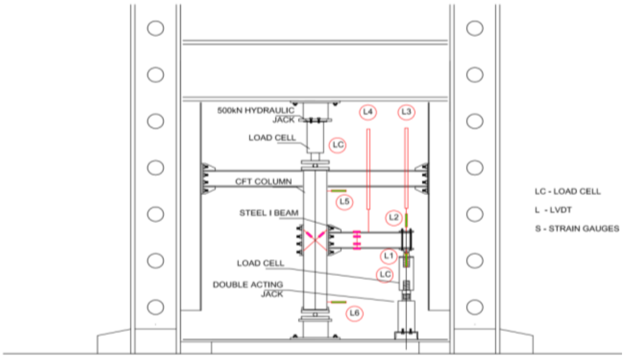

Fig.3. Schematic diagram of test setup

Fig.4. Test Arrangement Photograph

The loading protocol was chosen as ANSI/AISC 2002 cyclic loading programme as shown in Fig. 5 [13]. The rotation mentioned in the protocol was converted as tip displacement of the beam and was monitored in real time. Vertical cyclic load corresponding to these displacements were captured and stored continuously.

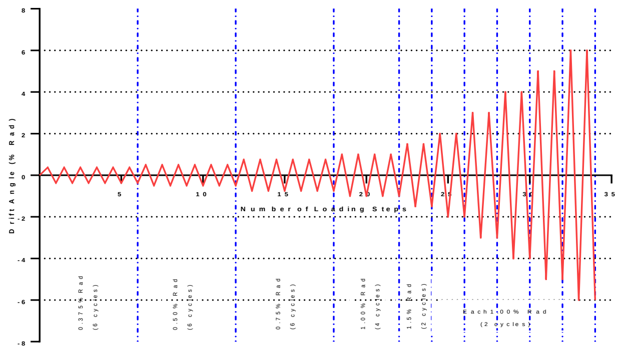

Fig.5. Loading protocol

III. EXPERIMENTAL RESULTS AND DISCUSSIONS

The observed physical phenomenon and key structural response parameters captured during the experiment are presented and discussed in detail in the following sections.

A. General observations

The test specimen ESFF220 exhibited good ductile properties with characteristics of plastic hinge formation on beam near the vertical stiffener. Actual material test data was used to find out the plastic flexural strength of beam (Mp). In order to calculate moment, force at the tip of beam multiplied by the distance between face of the column to the loading line were used. The initial cycles were perfectly elastic. During 0.02rad cycle of loading peeling of surface paint on the beam near the stiffener region was observed. Slight buckling of top flange was observed during the second cycle of 0.05 rad and which almost get flattened over the negative cycle. Weld failure occurred during the first cycle of 0.06rad and the load dropped to 90% of the peak load. The experiment stopped at an angular rotation of 5.663%rad on the negative side. Second Specimen ECSC220 also showed similar ductile behaviour. When the angular displacement reaches 0.03rad cracks perpendicular to the beam length originating from top and bottom flange location were visible in the painted area of beam near the vertical stiffener. During the second cycle of 5%rad specimen showed drop in moment capacity with slight torsional buckling which may be due to non uniform distribution of stresses between end plate and steel beam. For both the specimens peak minimum rotation observed was greater than 0.04 rad in both positive and negative cycles, which satisfy the guidelines provided by AISC 341-10 for composite special moment resisting frame. Hence this type of semi through type connection can be utilized for seismic region.

B. Moment Rotation relation

The hysteretic loops of moment at the column face and angular rotation in % rad is presented in Fig.6(a) and 6(b). The total rotation of the structure is contributed by beam, column and panel zone. From the experimental data evaluation of local and total rotation it was clear that major contribution was from beam because of the design philosophy, strong column strong joint and week beam. The result shows a stable hysteretic pattern with increase in loop area for both the specimen.

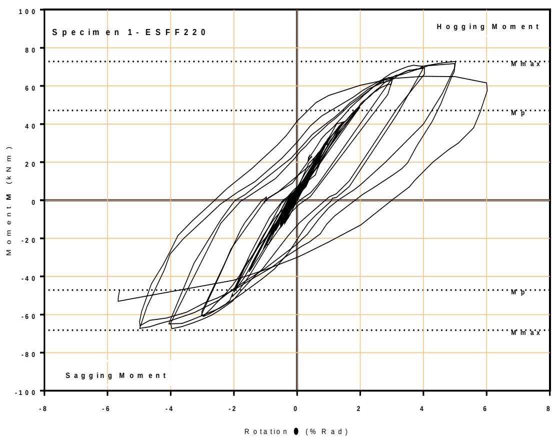

Fig.6(a). Moment Rotation hysteresis curve (1ESFF220)

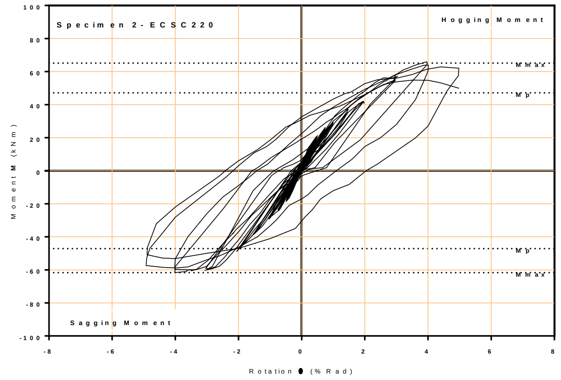

Fig.6(b). Moment Rotation hysteresis curve (2ECSC220)

Envelope curve of the moment rotation relationship were extracted from hysteretic curve and plotted in Fig. 7 for detailed investigation on stiffness comparison. The experimental results of flexural strength and comparison with beam plastic moment capacity are listed in Table III. Beam plastic moment capacity (Mp) was calculated based on the true strength observed from coupon test.

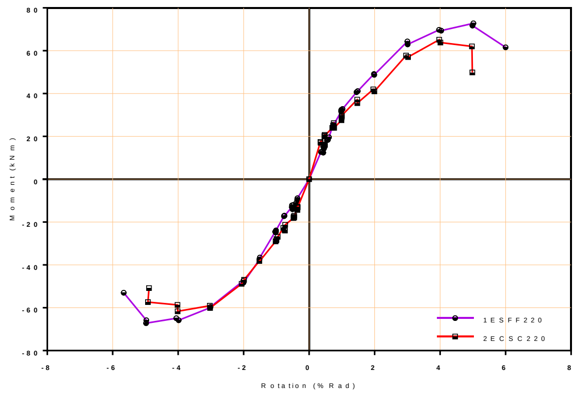

Fig.7. Moment – Rotation Envelop curve

General yield point method was adopted to find the yield point from the skeleton curve. The ductility of the joint can be evaluated by using displacement ductility coefficient μ, given by the ratio of ultimate displacement, Δ_u to the yield displacement Δ_y. The yield point was corresponding to 45.57MPa moment for 1ESFF220 and 43.09MPa for 2ECSC220.

$$\mu = \frac{\Delta_u}{\Delta_y} \tag{1}$$

For the first specimen displacement ductility coefficient calculated was 4.36 while specimen 2 was having higher value of 5.27 even though total drift angle observed was lesser by 1% rad. Maximum displacement was chosen corresponding to last successful cycle before failure whose moment resisting capacity of specimen does not fall below beam plastic moment capacity M_p.

Table III. Test Results

Specimen	Peak Moment (MPa)		M_{test}/M_p		Initial stiffness (kNm/Rad)	
	+Mmax	-Mmax	Hogging	Sagging	Hogging	Sagging
1ESFF220	72.813	67.221	1.544	1.425	33.02	24.66
2ECSC220	65.126	61.664	1.380	1.308	44.80	34.09

The initial stiffness of the circular specimen 2ECSC220 was higher than that of square CFT specimen by 35.7% and 38.24% respectively on hogging and sagging.

C. Energy Dissipation capacity

Energy dissipation can be found by calculating area bounded by each loop of hysteretic curve. Cumulative energy dissipation is plotted in Fig. 8. Plot shows remarkable increase in energy dissipation capacity after reaching 0.01rad. This parameter gives a clear picture of rate of stiffness degradation and deformation occurred during the increment of rotation cycles. Both the specimen follow almost same path which indicate that split bolt assembly is comparable to conventional straight bolt configuration.

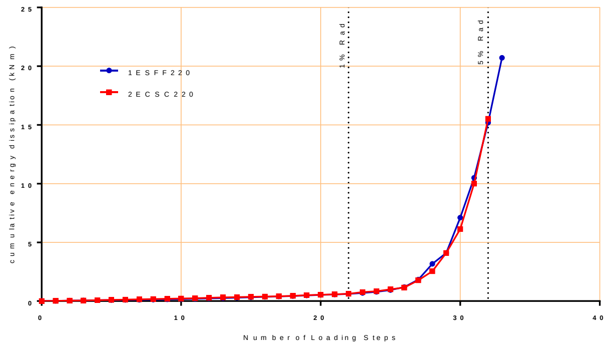

Fig.8. Cumulative Energy Dissipation

D. Failure Mode

During the first cycle of 6% rad. there was a breaking sound followed by initiation of weld crack occurred between top flange and endplate. After that there was no increment in loading but rotation was increasing. Once the load was reduced to 85% of the ultimate load, experiment terminated. On removal of load the beam shows permanent overall deflection and slight local flange buckling at plastic hinge location. The photograph of the joint region after testing is shown in Fig.9. The confinement effect of prestress induced by bolt tightening increases the strength and stiffness of panel zone, which intern reduces the chance of panel zone failure at seismic regions. No inelastic deformations were observed at the endplate and predrilled hole locations on the column as in case of failures reported in surface bolted connection [14]. Photograph showing the plastic hinge formation and weld failure is marked in Fig. 9.

Fig.9. Photograph of specimen after test

For circular specimen experiment was terminated at

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second cycle of 5% rad were moment resistance tends to fall below Mp and observed minor torsional buckling of beam which leads to non uniform distribution of stress at joint. There was difference between through bolt and split bolt on maintaining the prestress force during the cyclic loading. Split bolt assembly strength depends on the bolt lap length and bond between bolt and concrete which usually reduce during cyclic loading. After debonding the resistance to bending and axial pull/push depends on the tread capacity of the bolt and split assembly.

IV. CONCLUSIONS

The proposed connection allows to completely avoid field weld. So the erection speed can be increased, thus reducing construction uncertainties. The presence of triangular stiffener is effective in reducing the local distress like prying on end plate and successfully moves the plastic hinge away from the face of the column which reduces early stage failures of joint. The test structure response indicate that connection using through bolt can achieve good seismic performance by incorporating the participation of the panel zone in resisting the story drift without physical damage for both circular and square specimens. The specimen shows good hysteretic behaviour with enough energy dissipation capacity and stable rotation more than 0.04 rad. The joint can be classified as composite special moment resisting frame as per AISC 341-10. Split bolt assembly configuration proves to be practically a good choice for circular column joint without compromising the seismic performance when compared to conventional straight through bolt in square sections. The total rotation of the connection is largely contributed by the bending of beam by yielding. Also stiffness degradation was predominant when compared to strength degradation, which ensure the elimination of brittle failures.The failure mode observed was weld cracking at the beam flange - endplate connection region and shows ductile failure pattern with better deformation ability when compared to surface bolting.

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