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Behaviour of bolted connections in concrete-filled steel tubular beam-column joints

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Abstract. Many authors have established the usefulness of concrete filled steel tubular (CFST) sections as compression members while few have proved their utility as flexural members. To explore their prospective as part of CFST frame structures, two types of connections using extended end plate and seat angle are proposed for exterior joints of CFST beams and CFST columns. To investigate the performance and failure modes of the proposed bolted connections subjected to static loads, an experimental program has been executed involving ten specimens of exterior beam-to-column joints subjected to monotonically increasing load applied at the tip of beam, the performance is appraised in terms of load deformation behaviour of joints. The test parameters varied are the beam section type, type and diameter of bolts. To validate the experimental behaviour of the proposed connections in CFST beam-column joints, finite element analysis for the applied load has been performed using software ATENA-3D and the results of the proposed models are compared with experimental results. The experimental results obtained agree that the proposed CFST beam-column connections perform in a semi-rigid and partial strength mode as per specification of EC3.

Keywords: concrete filled steel tubular columns; extended end plate (connection); seat angle (connection); through bolts; monotonic loading

1. Introduction

Steel tubes infilled with concrete form composite members, are referred as Concrete-Filled Steel Tubes (CFST). CFST columns demonstrate outstanding structural behaviour including high strength, large stiffness and ductility at the same time these structures enhance aesthetics and constructional benefits. The steel tube offers confinement and improves the stiffness and strength of the infilled concrete while eliminating the use of formwork in construction. The concrete decreases the probability of local bucking of the steel tube. These advantages of CFST sections have been recognized in contrast with conventional concrete and steel structures, have directed to a rising practice of CFST columns in recent tall buildings (Shams and Saadeghvaziri 1997). Many studies have been carried out to investigate the behaviour of CFST columns subjected to various types of loadings.

Furlong (1967), Knowles and Park (1969) and Tomii *et al.* (1977) are some of the earliest researchers who studied the behaviour of concrete filled steel tubular columns subjected to concentric compression. In past two decades, elaborated experimental and theoretical studies performed by many researchers like Schneider (1998), Uy (2001), Han (2002), Liu *et al.* (2003), and many others have established their usefulness. Bahrami *et al.* (2011) investigated the nonlinear analysis of concrete-filled steel composite columns subjected to axial loading to predict the behaviour of the columns and ultimate load capacity, using software LUSAS. Huang *et al.* (2012), carried out different experiments to study the structural performance of concrete filled circular steel tube columns subject to four concentric loading schemes. Then, a generalized prediction method is developed to evaluate the ultimate load capacity of CFST columns subject to various loading conditions.

Many researchers have investigated the concrete filled tubular sections subjected to flexural loads and proved their effectiveness. Zhang and Brahmachari (1994), Elchalakani *et al.* (2001), Zhao (2002), Han (2004) and Prabhavathy *et al.* (2006) conducted experiments on concrete filled steel tubes subjected to flexural load and obtained that filling of concrete in steel hollow tube increased the flexural strength which further delays the failure of the columns. Kang *et al.* (2007) investigated the flexural behaviour of concrete-filled steel tubes, with variety of the filling material and concluded that CFST beams have good ductility. Arivalagan *et al.* (2008, 2010) experimentally compared the ultimate moment capacity and behaviour of unfilled and concrete-filled rectangular hollow tubular columns to investigate the effect of different parameters on moment carrying capacity of concrete-filled RHS beams. Arivalagan (2010) also presented nonlinear finite element (FE) model for concrete–filled RHS section used as a beam. Sundarraja (2012) carried out an experimental study to investigate the suitability of carbon fibre reinforced polymer (CFRP) to

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strengthening of CFST members under flexure. A non-linear finite element model was developed using the software ANSYS 12.0 to validate the test results. Valsa Ipe *et al.* (2013) concluded from the experimental investigations that the strength to weight ratio of composite box beams is much higher than reinforced cement concrete (RCC) beams and ductility index is also more than RCC and empty beams.

The results of mentioned studies conclude that filling of steel tube with concrete enhances the flexural strength and moment carrying capacity which is because of increase in moment of inertia and moment resistance of the section contributed by steel section. The increase in stiffness controls the local buckling of the steel tube also. Kumari *et al*. (2017) utilized guided waves to monitor degradation in CFST sections.

So, in the present study CFST members are used as both beam and column to explore their potential in CFST framed structures. Square/rectangular concrete filled steel tubular sections as beams and columns have been selected for study because it's a proven fact that the rectangular sections are preferred over their circular counterparts by the designers due to architectural reasons and most importantly more convenient connectivity between the beam and column. Beam-column connections being an important part of any structure, are focus of this study.

The design of beam-column connections is of significant importance for the economy of the structure as they involve a big amount of the cost of structure Also the beam-column connections constitute a vital part of a structure, hence are designed more conservatively than other components. The reason behind is the complexity to analyze these connections which is more than individual members and the deviancy between analysis and actual behaviour is large.

Additionally, in case of overloading, the failure confined to individual members is desired rather than in connections, which can affect many other members also. According to the literature available, a wide array of beam-CFT column connections has been studied over the past few decades. The most convenient connection involves attaching the steel beam directly to the outer surface of the steel tube for simple connections according to Dunberry *et al.* (1987) and Shakir *et al.* (1995). Alostaz *et al.* (1996) and Elremaily *et al.* (2001) investigated seismic performance of two categories of connection details; through column and through beam. Beams passing through panel region are mentioned as through beam connections. The experimental results disclosed that these connections enhanced the seismic behaviour. But due to its complicated behaviour in the panel zone these connections may create difficulty in erection whereas through column connections can help the field construction. Fujimoto *et al.* (2003) also investigated the connection details to enhance its seismic performance.

End plate type bolted connections are extensively used connections in steel structures (Kaushik *et al.* 2013) and are also used as moment-resistant connections. Due to simple fabrication techniques and speedy construction have made these connections the most popular systems of connecting members of steel frames. The behaviour and analysis of bolted end plates is quite complex in spite of their easy use. Many researchers have studied end plate connections. Kim *et al.* (2007) introduced the four types of FE model in order to investigate the best modeling technique for the structures with bolted joints. Diaz *et al*. (2011) presented a 3D FE model of steel beam to column extended end plate joint to obtain its behavior. Wang and Guo 2012 carried out an experimental study on the structural behavior of blind bolted end plate connections between CFST columns and steel beams under monotonic loading to develop an easy to use bolted moment connection in the thin-walled structures.

The proposed connection was evaluated for the failure modes, moment– rotation relationship and connection rigidity. Kaushik et al. 2013 studied the fatigue behaviour of bolted beam to column end-plate connection subjected to static loading. Wang and Spencer (2013) conducted experimental tests and numerical analyses to study the behavior of the blind bolted end plate joint with CFST columns to steel beam. The behaviour was evaluated by testing four full-scale sub-assemblages representative of interior or exterior beam-to-column joints. Another type of connections discussed in literature is seat angle connections. Seat angles are attached to beam section on both sides and these angles are bolted to the CFST column. These connections have benefit of easy and fast assemblage and better-quality control than welded connections. Pirmoz *et al*. (2008) studied the behaviour of bolted top-seat angle connections with web angles using 3D parametric models.

Most of the previous studies are based on CFST column and steel beams (not concrete filled). No study of the behaviour of CFST column connected to CFST beams is available. In the present study, two types of connections; extended-end plate and seat-angle connections using through length bolts and normal high strength bolts are investigated. Total ten sub-assemblages (five with extended end plate and five with seat angle) of composite joints that replicate the external region have been subjected to monotonic loading. Various parameters considered are the length and diameter of bolts used. Furthermore, FE models have been developed to simulate the performance of the bolted end plate and seat angle connections for CFST beam column joints to validate the experimental investigations.

The finite element program ATENA-3D has been employed in the analysis. The concrete confinement effects, interface between the concrete core and steel tube, the end plate and steel tube of column have been considered. Based on the present study, the fundamental behaviour of the joints in terms of strength has been evaluated and the viability of connecting an extended end plate and seat angle to the CFST beam with CFST column using through bolts and normal high strength bolts to improve the joint behaviour has been explored.

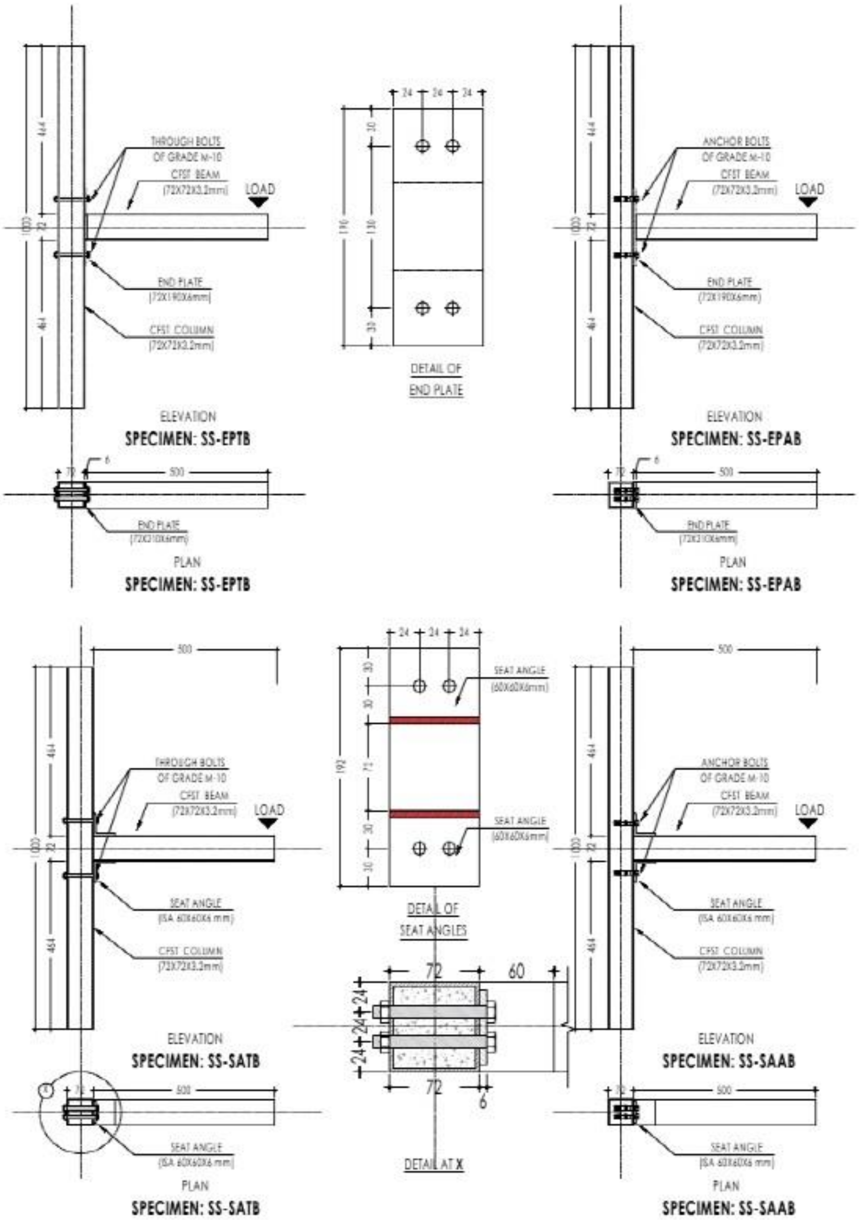
2. Experimental study

2.1 Material properties

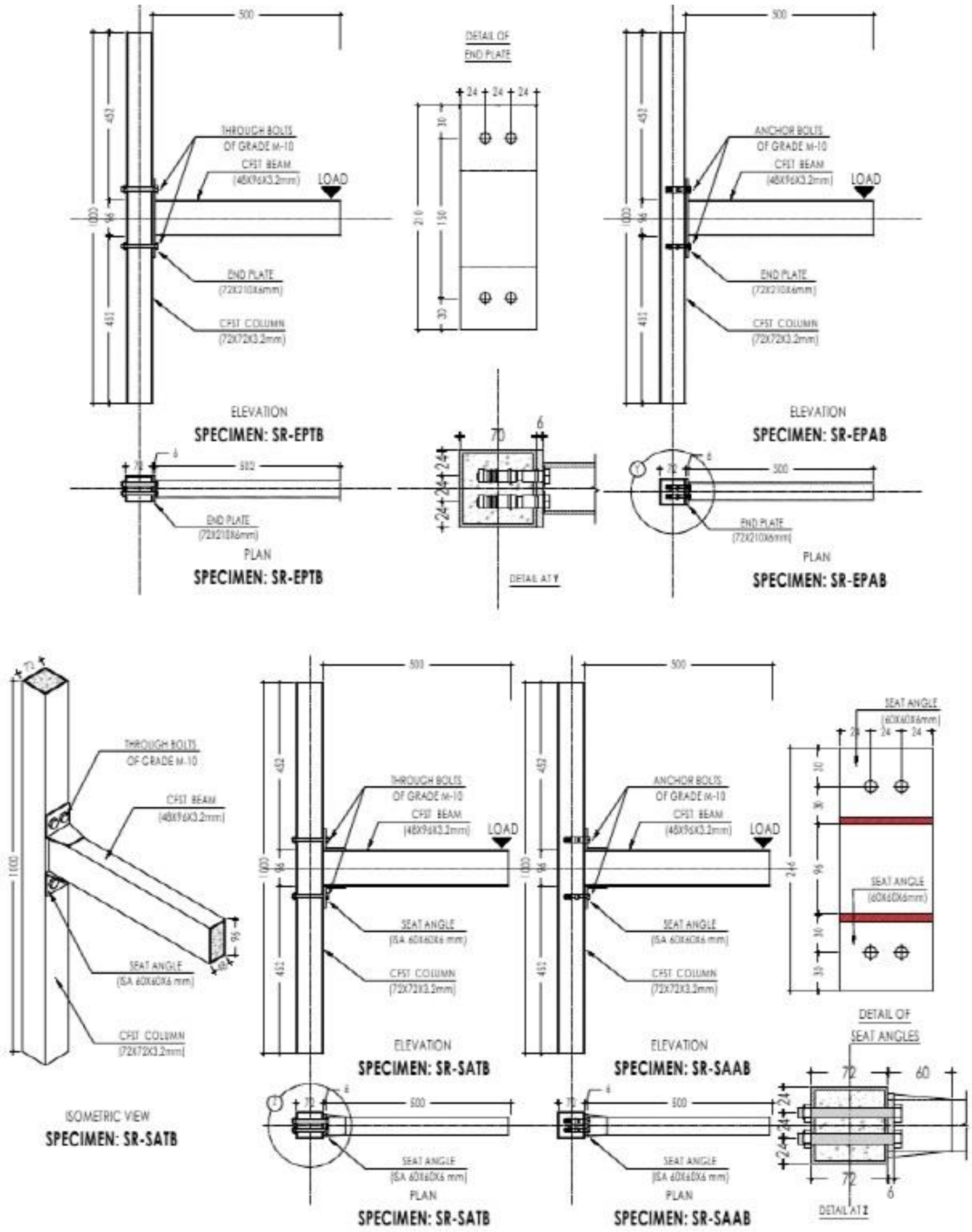
Steel hollow section: Square and rectangular steel hollow sections of TATA Structura make of Grade Yst-240 (with minimum Yield strength 240 N/mm^2) conforming IS 4923, have been used in the experiment. Modulus of

Table 1 Detail of test specimens

Sr. No.	Specimen Label	Detail of Specimen
1.	SS-EPTB	Square column Square beam- End plate through bolted
2.	SS-EPAB-1	Square column Square beam - End plated anchor bolted (10 mm dia bolts)
3.	SS-EPAB-2	Square column Square beam - End plate anchor bolted (8 mm dia bolts)
4.	SS-SATB	Square column Square beam - Seat angle through bolted
5.	SS-SAAB-1	Square column Square beam - Seat angle anchor bolted (10 mm dia bolts)
6.	SS-SAAB-2	Square column Square beam - Seat angle anchor bolted (8 mm dia bolts)
7.	SR-EPTB	Square column Rectangular beam- End plate through bolted
8.	SR-EPAB	Square column Rectangular beam - End plate anchor bolted (10 mm dia bolts)
9.	SR-SATB	Square column Rectangular beam - Seat angle through bolted
10.	SR-SAAB	Square column Rectangular beam - Seat angle anchor bolted (10 mm dia bolts)



(Note: All Dimensions are in mm and drawings are NTS)
Fig. 1 Design details of SS-EPTB, SS-EPAB, SS-SAAB connections



(Note: All Dimensions are in mm and drawings are NTS)
Fig. 2 Design details of SR-EPTB, SR-EPAB, SR-SATB and SR-SAAB connections

elasticity of steel, E_s is taken as 2.1 x10^5 N/mm^2. In coupon tests, average yield strength has been found to be 260 N/mm^2.

Core concrete: Concrete of grade M-25 has been used for the test specimens. The average 28 days' compressive strength of the concrete has been found to be 35.5 N/mm^2. The equivalent cylinder strengths f_c' has been determined as being 28.4 N/mm^2. The IS 456:2000 stress-strain relationship has been used for the purpose of calculating the basic material properties (as given below) used in analysis.

Modulus of elasticity of concrete, E_C = 29790.9 N/mm^2, Characteristic strength of concrete f_{ck} = 25.0 N/mm^2.

Bolts: High strength bolts (conforming IS: 1364:2003) of the Grade 10.9 M-10 having ultimate strength and yield strength of 1000 N/mm^2 and 900 N/mm^2, respectively, have been used in all samples. Two sub assemblages have been connected with bolts of the Grade 8.8 M-8 having ultimate strength and yield strength 800 N/mm^2 and 640 N/mm^2, respectively. To ensure bond behaviour, a pilot study has been conducted for joints using normal concrete mix but occurrence of bond slip has been observed at lower loads

Table 2 Summary of test specimen data

Specimen Label	Column section b x h x t (mm)	Column height (mm)	Beam Section b x h x t (mm)	Beam Length (mm)	End plate Size b x h x t (mm)	Bolt dia (mm)	Seat Angle size (mm)
SS-EPTB	72 X 72 X3.2	1000	72 X 72 X3.2	500	72 X 190 X 6	10	
SS-EPAB-1	72 X 72 X3.2	1000	72 X 72 X3.2	500	72 X 190 X 6	10	
SS-EPAB-2	72 X 72 X3.2	1000	72 X 72 X3.2	500	72 X 170 X 6	8	
SS-SATB	72 X 72 X3.2	1000	72 X 72 X3.2	500	-	10	60 X 60 X 6
SS-SAAB-1	72 X 72 X3.2	1000	72 X 72 X3.2	500	-	10	60 X 60 X 6
SS-SAAB-2	72 X 72 X3.2	1000	72 X 72 X3.2	500	-	8	50 X 50 X 6
SR-EPTB	72 X 72 X3.2	1000	48 X 96 X3.2	500	72 X 210 X 6	10	
SR-EPAB	72 X 72 X3.2	1000	48 X 96 X3.2	500	72 X 210 X 6	10	
SR-SATB	72 X 72 X3.2	1000	48 X 96 X3.2	500	-	10	60 X 60 X 6
SR-SAAB	72 X 72 X3.2	1000	48 X 96 X3.2	500	-	10	60 X 60 X 6

especially in case of bolts of smaller anchorage length. Hence to ensure proper bond of anchor bolt with concrete, Injection mortar FIS-V (Fischer brand) has been used. This is an approved material suitable for rebar connections in Concrete with compressive strength between 20 N/mm^2 to 60 N/mm^2.

2.2 Description of test specimens

The test specimens have been constructed in a T-shape to simulate the external region of a semi-continuous frame. Ten beam-to-column connections have been tested under similar loading. Tables 1 and 2 summarizes the detail of the specimens. Figs. 1 and 2 provide the design details of the proposed beam-to-column connections. The columns for all specimens have been concrete filled square steel tubes of 1000 mm height with cross-section 72 X 72 X 3.2 mm, whereas, the beams of two types i.e., concrete filled square steel tubes of 500 mm length with cross-section 72 X 72 X 3.2 mm in specimens named as SS-EPTB and SS-EPAB-1 and SS-EPAB-2 and rectangular steel tubes with cross-section 48 X 96 X 3.2 mm in specimens SR-EPTB and SR-EPAB have been used (Fig. 2). End plates and seat angles have been connected to CFST beams with the help of fillet weld of 6 mm size.

2.3 Experimental set-up

The arrangement of the test set-up is shown in Fig. 3. In order to investigate the behaviour of the bolted connections under monotonically increasing loads, the loading has been applied at the beam end in the vertical direction by a hydraulic jack of 600 kN capacity. Before the application of load, all the measurement networks have been checked to record the initial readings in the preloading phase. Then, the loading jacks have been increased by 0.2 kN and after a while set to zero before starting formal loading. In the loading phase, the test has been performed in the load control mode. Test has been performed till the maximum load achieved. The values of displacement have been automatically recorded by two Linear Variable Displacement Transducers (LVDTs) acting on the beam tip and 100 mm away from tip.

3. Numerical modelling of CFST beam-column joints

The finite element method (FEM) is a widespread tool to validate and envisage the actual behaviour of complex structures. In past few years, extensive general-purpose finite element codes with more and more perfect functions have developed speedily which has made the application of FE analysis possible on different kinds of bolted connections.

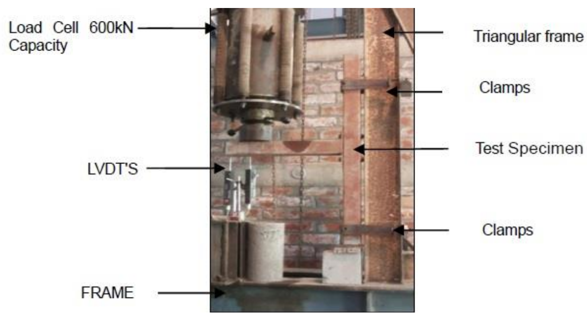
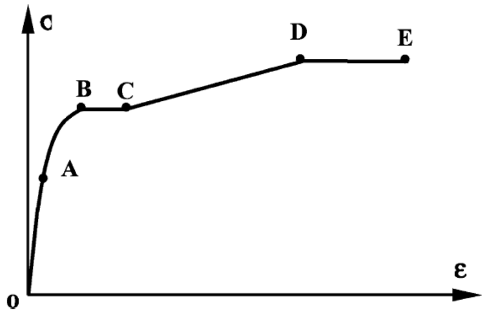
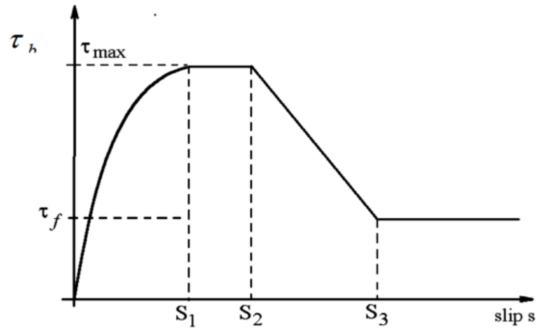


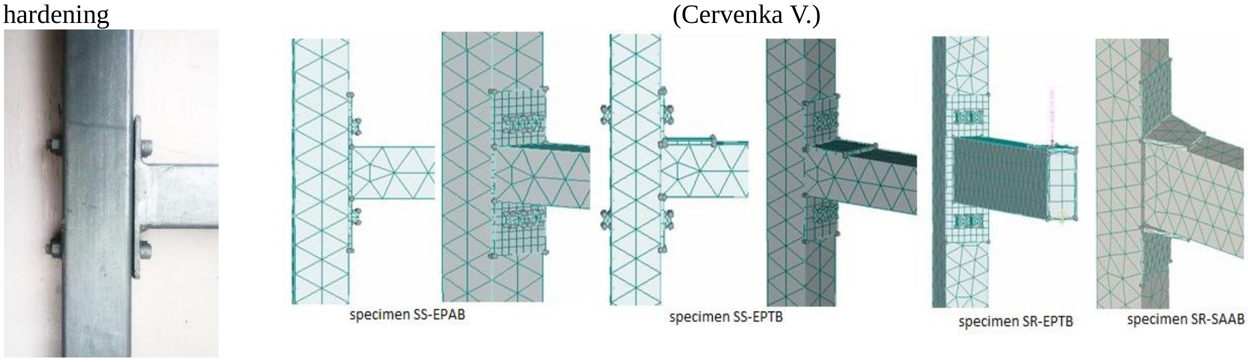
Fig. 3 Experimental setup



(a) Typical stress–strain curves for steel with strain hardening



(b) Bond-slip law by CEB-FIB model code 1990 (Cervenka V.)



(c) Experimental specimen (SS-EPTB) and FE Modelling of various specimens

Fig. 4 FE Modelling of various beam column joints

In the present study, a numerical model has been developed in ATENA-5.4-1, a general-purpose finite element code with numerous distinctive features for non-linear analysis of reinforced concrete and other composite structures. The nonlinear behaviour of CFST beam-column joints under the static load has been carried out by employing FE modelling and load carrying capacity of exterior concrete-filled steel hollow tubular section (CFST) beam-column joints has been studied. The response and failure modes have also been included in investigations. Various modelling related problems viz. the boundary conditions, interaction between concrete-steel tube, representation of different materials etc. are investigated using a broad parametric study. In the analysis, the interaction between column wall and the end-plate, geometric and material nonlinearities have been considered by an appropriate method. Then the proposed model is validated by associating the results with experimental results. The loading resistance of these connections, and the influence of connection details on the connection behaviour has been discussed on the basis of the results obtained from FEA.

3.1 Concrete

To model the infilled concrete in ATENA 3D, a solid brick element with minimum eight and maximum twenty nodes is taken. This element has three degree of freedom (X, Y, Z) directions at each node. It is also accomplished of capturing cracking and plastic distortion in each direction (Cervenka V.) The effect of confinement (increase in the compressive failure stress due to compressive stresses in other directions) is considered automatically by the material models available in Atena. A more detailed sensitivity towards confinement is executed in the Non-linear Cementeous-3 model which considers multidimensional failure surfaces in the material model.

3.2 Steel tube

The hollow steel tube is modelled by Von-Mises plasticity model. The Von-Mises yield criterion flow rule characterizes the correct description of the plastic behaviour of metals (Chakrabarty 1998). So, an elastic-plastic model 3D-Von Mises yield criterion (refer Fig. 4) has been adopted to describe the constitutive behaviour of steel tube to analyse the behaviour of the steel in the plastic hardening range. For meshing of steel tube tetra or brick and tetra (mixed meshing) element can be used.

3.3 Modelling of bolts

The bolt shank has been modelled as reinforced bar with bond and bolt head has been modelled as steel plate of very small size. The simplest way to model a bolt (headed anchor) is to trigger no slip for the bar end connected with head. For more precision, a combination of 2-D and volume element has been used in this analysis. Various interfaces need to be carefully handled. Hence the steel-concrete contact at shank area has been modelled as bond model. Properties of good bond have been given as bolts used were threaded bolts. The basic property of the reinforcement bond model takes the bond-slip relationship into account. The relationship describes the bond strength (cohesion) which depends on the value of bond slip between reinforcement and surrounding concrete. ATENA bond-slip model CEB-FIB model code 1990 (Fig. 4) has been considered. In this model, the laws generated are based on compressive strength of concrete, type and size of reinforcement and the confinement conditions (Cervenka V.). Interface between outer side of end plate and bottom side of steel plate (head) is simplified as no connection whereas shank top and head as perfect connection. Mesh refinement has been done for steel plate used as bolt head.

3.4 Interface modelling

The interface material model can be used to simulate contact between two materials such as for instance a joint between two concrete segments or a contact. In the tested connections, there are so many interfaces which need to be modelled. Though ATENA interface provides intelligent interface modelling but the work required to define the interface material parameters and related complications can be avoided by using perfect connection and no connection. Perfect connection is used to model interface for composite action of steel tube and core concrete as the focus of study being connection part. Interface between outer surface of CFST column and core concrete filled in CFST beam of joints with seat angles, have been modelled as no connection. Contact element has been used to model interface between end plate and outer of column steel plate. Connection between end plate and seat angle to CFST beam has been modelled as perfect connection.

3.5 Boundary conditions and Loading

In order to simulate the test set-up in proper way both ends of column part have been fitted with steel plates. CFST column is restrained against displacement in all three directions with the help of clamps. Monotonically increasing load is applied at steel plate modelled at the beam end. Values of concrete yield strength, concrete modulus of elasticity and steel yield strength and steel modulus of elasticity are entered in definition material type and analysis is run in pre-processor mode. Modelling of few connections has been illustrated in Fig. 4.

4. Analysis and discussions

4.1 Load displacement behaviour of extended end-plate bolted connections

4.1.1 Square beam with square column

Specimen SS-EPTB

Specimen SS-EPTB has rectangular end plate of 72 X 190 X 6 mm size, connected to the square CFST columns with the help of high strength through bolts of Grade 10.9 M-10. The load displacement curve obtained shown in Fig. 5, is fundamentally elastic before the displacement touched a level of 7.42 mm to a load value of 4.5 kN. A typical sound has been released from the connection zone at 28.14 mm (load 8.64 kN) displacement. It can be correlated with the bond failure of bolts in tension from the core concrete in column. The end plate of CFST beam started deviating from column face and the concrete around the shank started to loosen. This bond failure occurred in the column face directly in line with the beam top portion leads to the reduction of moment capacity (Fig. 5). Then again, the load starts increasing. The load attained is a maximum of 11.9 kN at displacement of 52.1 mm, and failure of the connection has been evident. The specimen SS-EPTB test has been terminated when signs of a larger deformation appeared. Moreover, the end plate has started yielding. The upper part of end plate has got inclined and departed from the column wall without showing signs of deformation in the column wall. No sign of local deformation of beam portion of the connection has been observed.

Specimen SS-EPAB-1

Specimen SS-EPAB-1 has 6 mm thick rectangular end plate of size 72 X 190 mm to the square CFST columns with high strength bolts of Grade 10.9 M-10. The load displacement curve of the beam (Fig. 5) has been observed to be same as that of specimen SS-EPTB. The deformation at the tip of beam corresponding to 10.5 kN load is about 47 mm and the specimen has reached a maximum load of 10.53 kN at displacement of 50.0 mm. The specimen SS-EPAB-1 test has been terminated at a displacement of 54.5 mm. The bolts in tension zone have shown slight anchorage failure from the column near the end plate of CFST beams and concrete around the shank area near the head of bolt lost its bond with concrete which further has led to the loss of moment capacity. The deformation of the end plate, which departed from the column face, is lesser in bending than that occurred in specimen SS-EPTB. The end plate in tension zone is deviated away by 4 mm from the column wall without causing any bending deformation of the column face-wall.

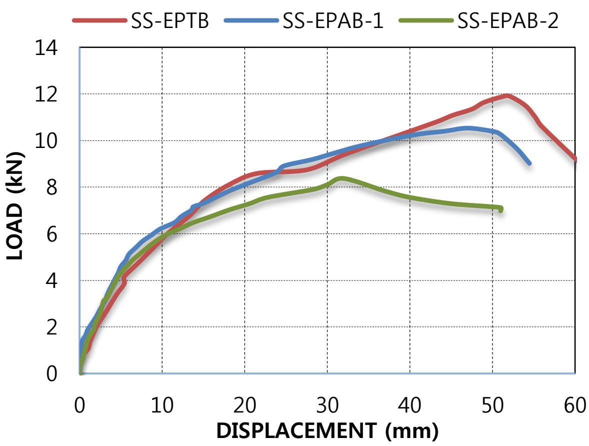


Fig. 5 Load- displacement relationship of specimen SS-EPTB, SS-EPAB-1 and SS-EPAB-2

Specimen SS-EPAB-2

Specimen SS-EPAB-2 has 6 mm thick end plate 72 X 170 mm joined to the CFST columns with bolts of Grade 8.8 M-8. The maximum load achieved is 8.37 kN at displacement of 32 mm, as shown in Fig. 5. After this point increase in displacement with decrease in load has been observed. In the test, no deformation in the column wall is detected. After the completion of test, the maximum deformation of the end plate has been found to be 5 mm and the end plate is deviated from the column wall similar to specimen SS-EPAB-1. Hence it can be concluded that size of plate, dia and grade of bolts considerably influence the capacity of the joint.

4.1.2 Rectangular beam with square column

Specimen SR-EPTB

Specimen SR-EPTB has rectangular end plate of 72 X 210 X 6 mm size, connected to the rectangular CFST beam to square CFST column with the help of high strength through bolts (120 mm long) of Grade 10.9 M-10. The load displacement curve (Fig. 6) of the beam is elastic at the displacement level of 3.42 mm against a load value of 4.34 kN. Distinctive cracking sounds have been produced from the connection zone at 5.7 kN load and 9 mm displacement due to start of bond failure of bolts from the column by the end plate of CFST beams and the concrete around the shank area. The failure mode is shown in Fig. 9. The displacement attained at tip of beam corresponding to the maximum load of 12.32 kN was 32.6 mm, and evident failure of the connection is found to be started. After achieving displacement of 42mm against load value of 12.12kN, drop in load has been observed. The specimen SS-EPTB test has been terminated at displacement of 52mm at the free end of beam. The end plate has started deforming and the upper part of end plate got separated from the column face with distinctive signs of bending deformation of end plate.

Specimen SR-EPAB

Specimen SR-EPAB has 72 X 210 X 6 mm rectangular end plate attached to the square CFST columns with high strength bolts of Grade 10.9 M-10. The load displacement curve (Fig. 6) of the test specimen is initially similar to that of specimen SR-EPTB, though the deformation mode of the end plate in specimen SR-EPTB is clearly different to that observed in specimen SR-EPAB. The specimen SR-EPAB achieved a maximum load value of 11.83 kN against a displacement of 17.5 mm. After this point the specimen SR-EPAB has shown continuous increase in deformation without any further increase in load. The tension bolts have shown bond failure from the column by the end plate of CFST beams and concrete around the shank area adjacent to the bolt head, lost its bond with concrete. The maximum value of displacement obtained at the end is 50.5 mm and the bending deformation of the end plate, after deviation from the column face, is lesser than that occurred in specimen SR-EPTB. The end plate near to the tension bolts have been pulled away by 4 mm from the face of column. The upper part of end plate has got inclined and departed from the column wall with no sign of local failure in flange of beam.

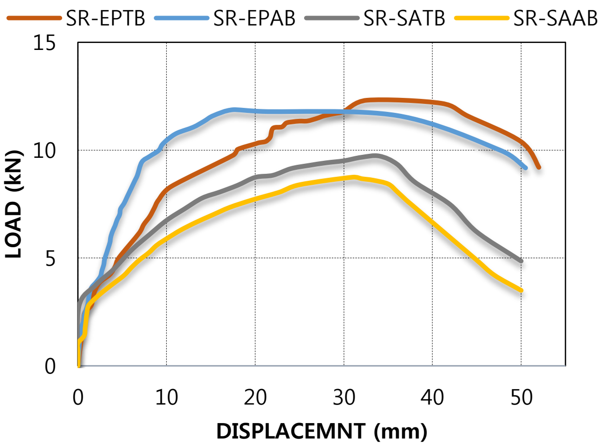


Fig. 6 Load- displacement of specimen SR-EPTB, SR-EPAB, SR-SATB and SR-SAAB

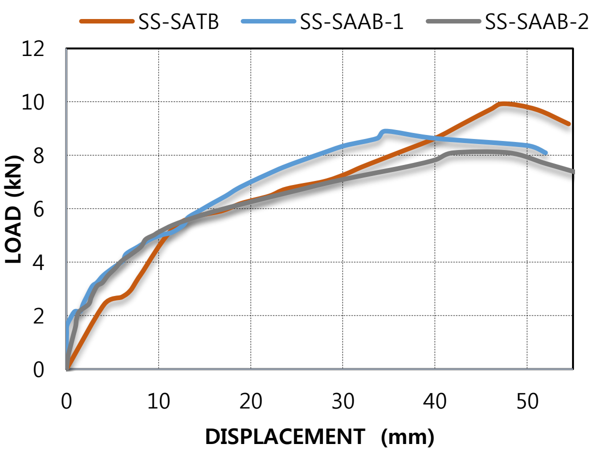


Fig. 7 Load displacement relationship of specimen SS-SATB, SS-SAAB-1 and SS-SAAB-2

4.2 Load displacement behaviour of Seat Angle bolted connections

4.2.1 Square beam with square column

Specimen SS-SATB

Specimen SS-SATB has been connected by seat angles of size 60 x 60 X 6 mm to both sides of CFST beam further connected to the CFST column with high strength through bolts of Grade 10.9 M-10. The maximum load of specimen SS-SATB has been more than that of specimen SS-SAAB-1 and SS-SAAB-2, as shown in Fig. 7. The specimen has attained a maximum of 9.936 kN at a displacement of 47 mm. The test has been stopped at a displacement of 54 mm and the maximum deviation of the connecting leg from the column wall is about 5 mm with noticeable crack at root of the seat angle

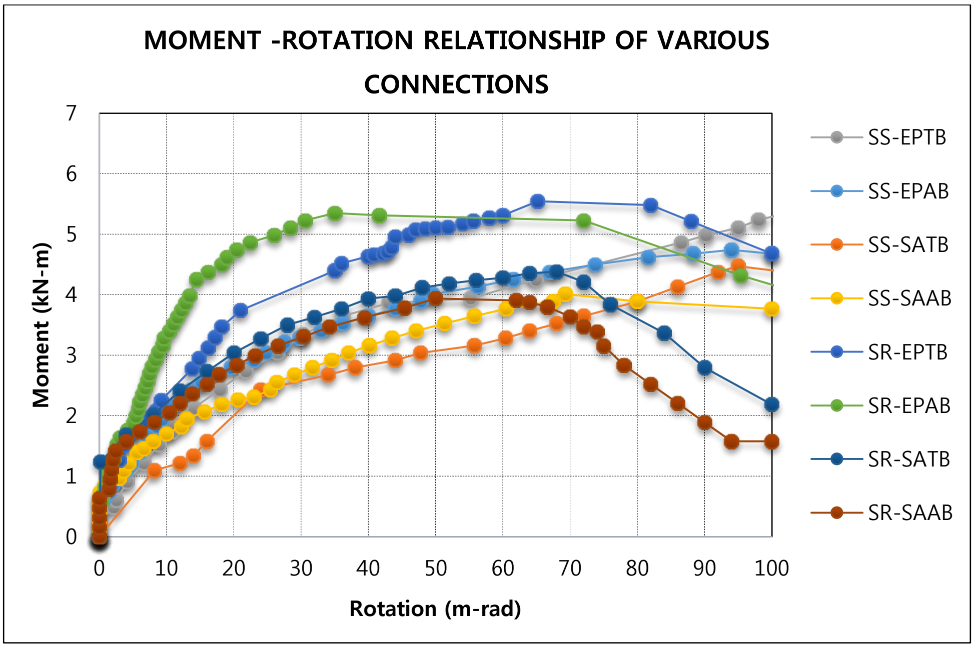


Fig. 8 Moment–rotation relationship of square column to square beam connections

Specimen SS-SAAB-1

Specimen SS-SAAB-1 has seat angles of size 60 x 60 X 6 mm connected to upper and lower part of beam further connected to the CFST column with high strength bolts of Grade 10.9 M-10. The load-displacement curve for the specimen (Fig. 7) initially have presented better behaviour then specimen SS-SATB. However, the deformations of the seat angle in specimen are noticeably larger than those obtained in specimen SS-SATB. The specimen SS-SAAB-1 has reached a load of 8.91 kN against displacement of 34.63 mm and the bolts in tension area displayed insignificant bond failure near the connecting leg of seat angle of CFST beams. The deformation at the free end of beam after 8.91 kN load has started increasing without any further increase in load. The connecting leg near the tension bolts has got yielded and dragged away by 4 mm from the column.

4.2.2 Rectangular beam with square column

Specimen SR-SATB

In specimen SR-SATB seat angles 60 x 60 x 6 mm and high strength through bolts of Grade 10.9 M-10, have been used. The maximum load of specimen SR-SATB has been more than that of specimen SR-SAAB, as shown in Fig. 6. The specimen has achieved a maximum of 9.73 kN at a displacement of 35 mm. The specimen SS-SATB test has been stopped at a displacement of 50 mm. The connecting leg has got deviation of about 5 mm from the column wall. Visible crack has been observed at root of connecting leg of the seat angle.

Specimen SR-SAAB

Seat angles of size 60 x 60 X 6 mm and high strength bolts of Grade 10.9 M-10 have been used in the test specimen. The load displacement curve for the connection has been plotted in Fig. 6. The specimen has shown almost similar behaviour to that of specimen SR-SATB. However, ultimate load of the SR-SAAB specimen is found lesser. The specimen SR-SAAB has achieved a maximum load of 8.75 kN at 31 mm displacement. The bolts have exhibited slight bond failure in tension area and concrete near the head of bolts lost its bond with concrete. A breakage at the root of the seat angle near to the bolts in tension and the connecting leg has been observed and is pulled away by 5 mm from the face wall of column.

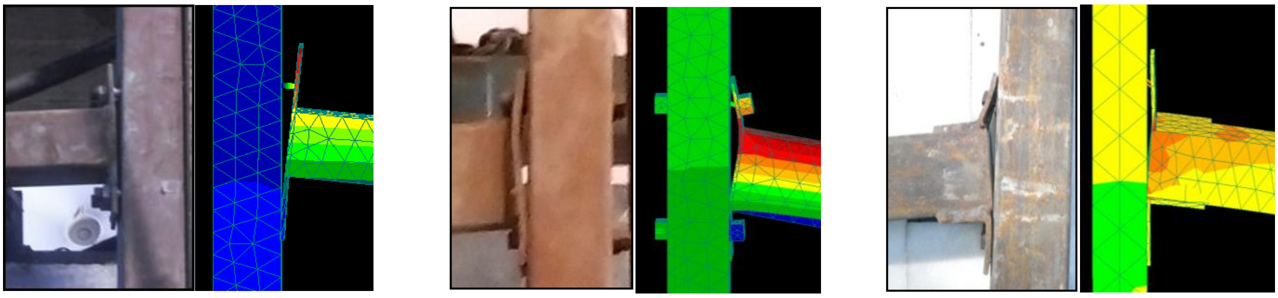
4.3 Moment- rotation relationships

M–θ_r curves are used to represent the relationship between the bending moment (M) and rotational angle (θ_r) of the beam-column connection. These curves characterize the behaviour of beam-column connections. The bending moment corresponding to the load P acting at free end is multiplied by the distance between the line of application of load and the end plate connected to the other end of beam. The connection rotation (θ_r) is defined as the variation in angle between the center lines of the beam and the column. It has been calculated with the help of displacement measured at the tip of beam. The moment–rotation curves for the connections are shown in Fig. 8.

In the connections with extended end plates, significant bending deformations occur in the end plate and the maximum deformations are experienced near the column face situated in the tensile area of the beam. The results have been tabulated in Table 3.

4.4 General remarks

Failure Modes: Based on the observations made during testing of various bolted connections, following failure modes have been witnessed:



(a) Bond failure of tension bolts in specimen SS-EPAB-1

(b) Yielding of end plate in specimen SR-EPTB

(c) Yielding of seat angle in specimen SS-SAAB-1

Fig. 9 Comparison of simulated and experimental failure modes

Table 3 Summary of measured test results

Sr. No.	Specimen Label	P_u (kN)	Δ (mm)	M_u (KN-m)	θ_{ru} (rad)	m	Θ
1.	SS-EPTB	11.90	52.1	5.36	0.104	0.76	4.34
2.	SS-EPAB-1	10.53	47	4.74	0.094	0.67	3.92
3.	SS-EPAB-2	8.37	32	3.77	0.064	0.61	2.67
4.	SS-SATB	9.936	47.5	4.47	0.095	0.73	3.96
5.	SS-SAAB-1	8.91	34.63	4.01	0.069	0.65	2.89
6.	SS-SAAB-2	8.1	42	3.65	0.084	0.59	3.50
7.	SR-EPTB	12.32	32.6	5.54	0.065	0.79	3.62
8.	SR-EPAB	11.88	20.8	5.35	0.042	0.76	2.31
9.	SR-SATB	9.73	34	4.38	0.068	0.62	3.78
10.	SR-SAAB	8.75	31	3.94	0.062	0.56	3.44

i. Deformation of the end plate: In case of through bolt connections
ii. Bond failure of bolts: In case of normal bolted connections
iii. Seat angle failure in seat angle bolted connections

Fig. 9 presents different deformation modes of few specimens tested. The deformation occurred in the end plate of specimen SS-EPTB is significantly more than that of specimen SS-EPAB and the ultimate load found to be more. The deformations in column wall due to the deviation of end plates are not observed but the end plates got yielded in all type of end plated connections. The deviation of the end plate for specimen SS-EPTB and SS-EPAB is 4 mm and 6 mm, respectively. The main reason of increased outward deformation of the end plate is due to pulling out of the tensile bolts. The observed results show that the minor cracking of concrete appeared in the vicinity of tension bolts in the column face. Any sign of bending of the bolts has not been observed during tests and the entire range of extended end plate connections have performed satisfactorily and better than seat angled connections where the connecting leg of seat angle yielded and even crack at the root has been observed in case of through bolts connection. In case of square column-rectangular beams connections, SR-EPTB have attained higher ultimate load than SR-EPAB. Square column-rectangular beams connections have shown 4-16% higher load carrying capacity over their square column-square beam counterparts.

It has been observed from the overall comparison of all the joints that through bolted connections with extended end plate performed better than the seat angle connections.

These connections have shown promising strength and ductility to be used as moment resisting connection. Various parameters viz. variation in size of end plate, dia and grade of bolts significantly affected the capacity of the joints.

4.5 Comparison of failure modes of experimental and FE results

After calculating and plotting iso-areas of normal stresses in the post-processor phase, the comparison of failure modes of experimental and simulated results is presented in Fig. 9. Here darker areas point toward compression and the lighter toward tensile stress, Part (a) of Fig. 9 represents bond slip failure mode of normal anchor bolted end plate connection, part (b) represents yielding of end plate near tension bolts in end plate connections using

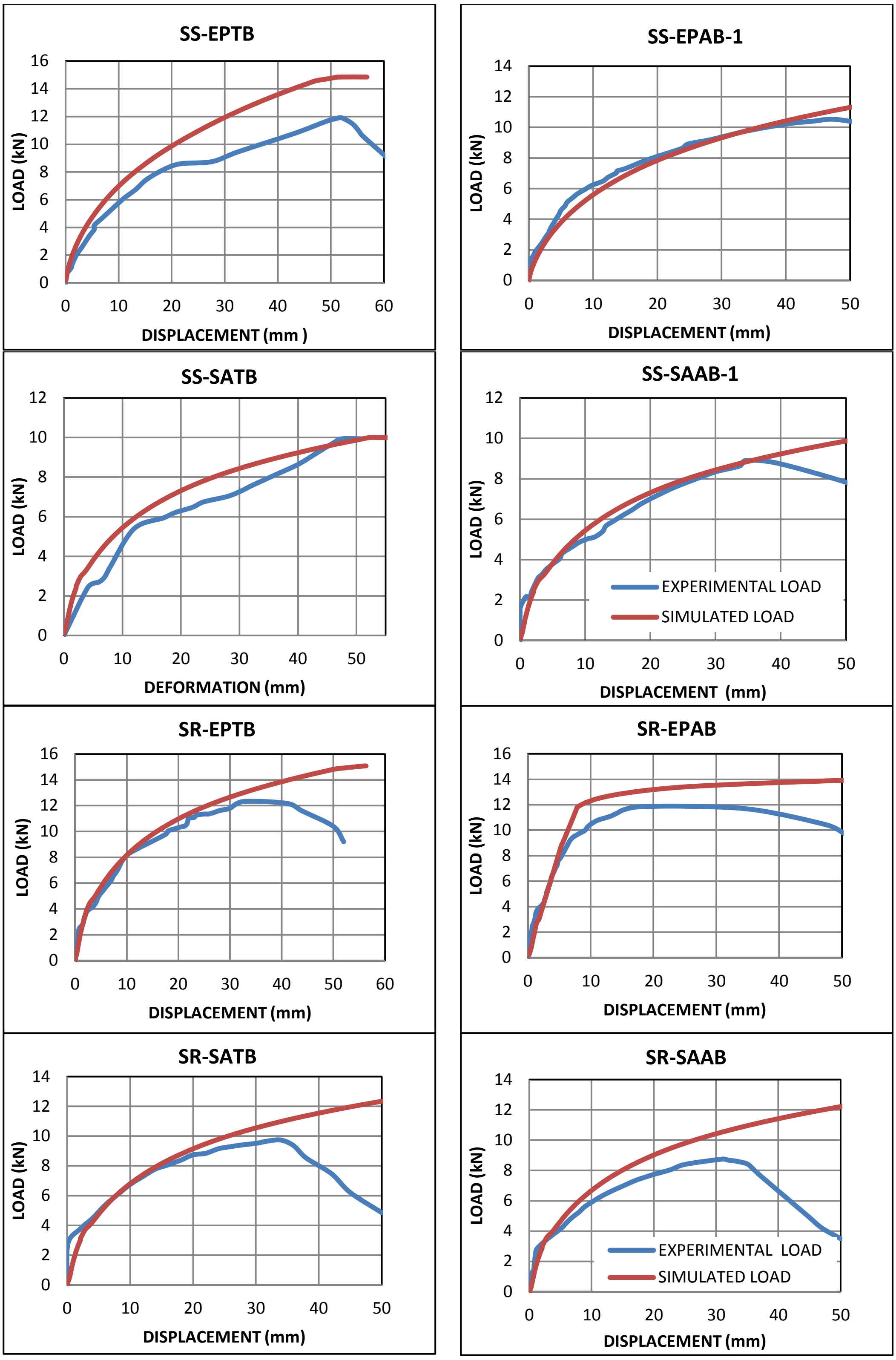


Fig. 10 Comparison of simulated and experimental load-displacement curves

Table 4 Classification of connections

Classification by rigidity	Classification by strength
Rigid, if $S_{j,ini} = k_b EI_b/L_b$ where k_b=8 for non-sway frames and k_b=25 for sway frames.	Full strength, if $M_u \geq M_{bp}$
Nominally pinned, if $S_{j,ini} \leq 0.5\, EI_b / L_b$ Where $S_{j,ini}$ - initial stiffness of the connection EI_b - flexural stiffness of the beam L_b - beam span.	Nominally pinned, if $M_{ur} \leq 0.25\, M_{bp}$ M_{bp} - plastic moment resistance of the beam
otherwise, it is semi-rigid.	otherwise, it is partial strength

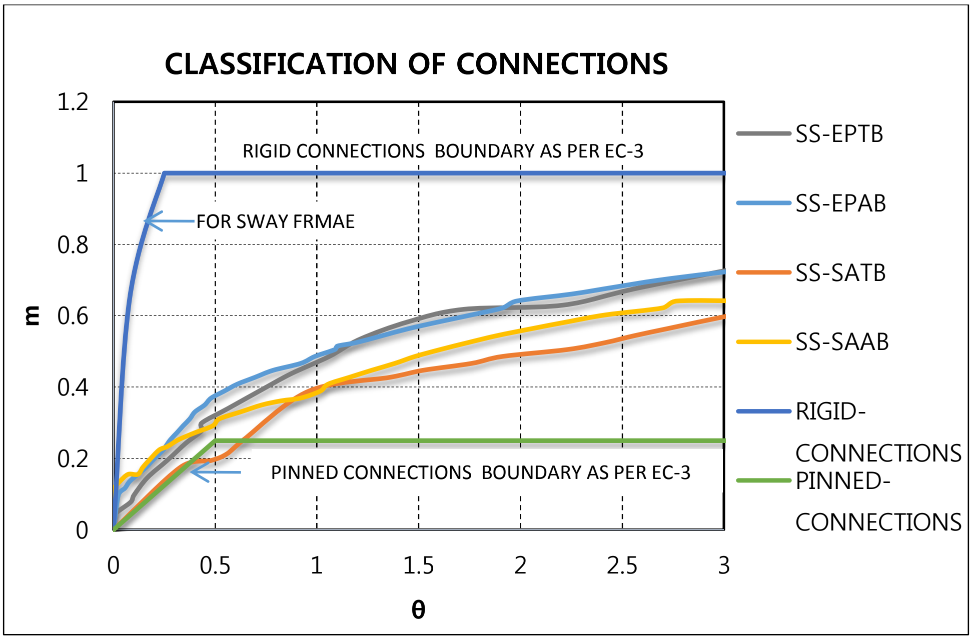


Fig. 11 Classification of Tested Connections square column to square beam

through bolts where as in part (c) failure mode of seat angle at root of seat angle anchor bolted connection, has been represented. Comparison of simulated and experimental Load-Displacement behaviour of various connections is presented in Fig. 10. The proposed FE models have predicted the behaviour of connections quite closely.

4.6 Classification of connections

In general, the connections are classified into following three categories on the basis of their behaviour:

i. Simple connections: Here the beam behaves as a simply supported beam as beam end is allowed to rotate freely. Shear and axial forces are transferred between the connecting members but bending moment is not allowed to be transferred.

ii. Rigid connections: In these connections rotation angle between beam and column remains constant. Bending moment along with axial and shear force is also transferred from beam to column.

iii. Semi-rigid connections: These category is designed to transmit shear force fully and partial transmitting of the bending moment from beam to the joint is allowed. The analysis of frames with such kind of joints is more complex.

The classification of connections has been made using the approach given in EC3 Part 1-8. It is done on the basis of rigidity and strength. Based on the comparison of initial stiffness and rotational stiffness of connection, a connection can be categorized as rigid, pinned, or semi-rigid (Wang *et al*. 2009).

Similarly, it can be classified in terms of strength by equating its moment resistance with the plastic moment resistance of the beam (refer Table 4).

M_{bp} is calculated as per EC4

$$M_{plrd} = m_{sq} \frac{h^2 b - (h - 2t)^2 (b - 2t)}{4} f_y \tag{1}$$

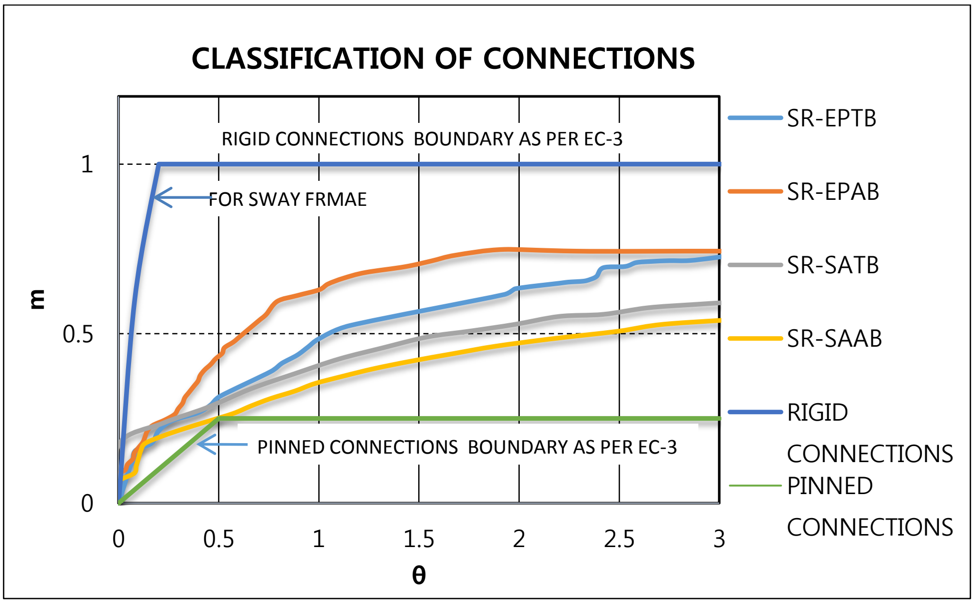


Fig. 12 Classification of Tested Connections of square column to rectangular beam

Herein M_{plrd} is plastic moment of beam; m_{sq} is a correction coefficient for different ratios of h/b and grades of concrete.

According to the approach discussed above, the classification of connections has been done from strength criterion and the analysis results for connections of sway frames are illustrated in Figs. 11 and 12. Here in, θ and m indicate dimensionless connection rotation and moment respectively detailed as under

$$m = \frac{M_u}{M_{bp}} \quad \text{and} \quad \theta = \frac{\theta_r}{M_{bp}} \cdot \frac{EI_b}{L_b} \tag{2}$$

Here in, θ_r is rotational angle corresponding to load and θ_{ru} (in Table 3) is rotational angle corresponding to ultimate load. It is observed that the majority of bolted tested specimens falls under the category of semi-rigid and partial strength connections.

5. Conclusions

In the present work, behaviour of CFST beam-column joints have been investigated experimentally as well as numerically. Based on the investigation, the following conclusions have been drawn:

1. The connections with extended end plate using through bolts have shown higher strength over seat angle through bolted connections. Specimen SS-EPTB has shown 19.76% higher strength over SS-SATB and Specimen SR-EPTB has shown 26.6% higher strength over SR-SATB.
2. The connections with through bolts have exhibited better strength over connections with bolts of normal length. Specimen SS-EPTB has shown 13.3% higher strength over SS-EPAB-1 and Specimen SS-SATB has shown 11.51% higher strength over SS-SAAB-1. Similarly, specimen SR-EPTB has shown has got 4.14% than SR-EPAB and SR-SATB has exhibited 11.2 % higher strength than specimen SR-SAAB.
3. Connections of rectangular beams of same cross section area as used in case of square beams with square columns using end plates, exhibited higher strength (3.5%-12.5%).
4. Specimen SS-EPAB-1 and SS-SAAB-1 with 10mm dia bolts has shown 25.7% and 10% higher strength over SS-EPAB-2 and SS-SAAB-2 using 8mm dia bolts respectively.
5. It has been observed that variation in size of end plate, dia and grade of bolts and bond strength of the matrix significantly affected the capacity of the joints.
6. The experimental results show that the proposed bolted connections behave as connections with partial strength as per the EC3 specification and display good stiffness and strength.
7. The proposed FE model is capable to predict the behaviour of connections closely.
8. The stiffness and strength of connections is enhanced with the increase in size of plate. It is expected that the strength of the bolted connections may be affected with change in various parameters such as thickness of end plate, the anchorage length of the bolt and cross-section of steel tube.

References

Alostaz, Y.M. and Schneider, S.P. (1996), "Analytical behaviour of connections to concrete-filled steel tubes", *J. Constr. Steel Res.*, **40**(2), 95-127.

Arivalagan, S. (2010), "Finite Element analysis on the flexural behaviour of concrete filled steel tube beams", *J. Theor. Appl. Mech.*, **48**(2), 505-516.

Arivalagan, S. and Kandasamy, S. (2008), "Flexural behaviour of concrete-filled SHS beams", *J. Civil Eng. Management*, **14**(2),

107-114.
Arivalagan. S. and Kandasamy, S. (2010), "Study on concrete–filled steel member subjected to cyclic loading", *J. Civil Eng. Management*, **1**(3), 458-465.
Bahrami, A., Badaruzzaman, W.H.W. and Osman, S.A. (2011), "Nonlinear analysis of concrete-filled steel composite columns subjected to axial loading", *J. Struct. Eng. Mech*., **39**(3), 383-398.
CEN. Eurocode 3 (2005), Design of steel structures Part 1-8: Design of joints. ENV 1993-1-8. Brussels: CEN.
CEN. Eurocode 4 (2004), Design of composite steel and concrete structures Part 1-1: General rules and rules for buildings. ENV 1994-1-1. Brussels: CEN.
Cervenka, V., Jendele, L., Cervenka, J., ATENA theory manual, Part-1, Prague.
Chakrabarty, J. (1998), Theory of Plasticity, 2nd Ed., McGraw-Hill Book Co, Singapore.
Dunberry, E., LeBlanc, D. and Redwood, R.G. (1987), "Cross-section strength of concrete-filled HSS columns at simple beam connections", *Can. J. Civil Eng*., **14**, 408-417.
Elchalakani, M., Zhao, X.L. and Grzebieta, R.H. (2001), "Concrete-filled circular steel tubes subjected to pure bending", *J. Constr. Steel Res*., **57**(11), 1141-68.
Elremaily, A. and Azizinamini, A. (2001), "Experimental behaviour of steel beam to CFT column connections", *J. Constr. Steel Res*., **57**, 1009-1019.
FEMA-350 (2000), Recommended seismic design moment-frame buildings. Federal Emergency Management Agency.
Fujimoto, T., Nishiyama, I. and Mukai, A. (2003), "Test results of CFT beam-to-column connection", *J. Constr. Steel Res*., **59**(2), 405-426
Furlong, R.W. (1967), "Strength of steel encased concrete beam-columns", *J. Struct. Division Proc. Am. Soc. Civil Engineers,* **93** (5), 115-130.
Han, L.H. (2002), "Tests on stub columns of concrete-filled RHS sections", *J. Constr. Steel Res*., **58**(3), 353-372.
Han, L.H. (2004), "Flexural behaviour of concrete-filled steel tubes", *J. Constr. Steel Res*., **60**(2), 313-337.
Huang, F., Yu, X. and Chen, B. (2012), "The structural performance of axially loaded CFST columns under various loading conditions", *Steel Compos. Struct*., **13**(5), 451-471.
Ipe, T.V., Bai, H.S., Vani, K.M., Zafar Iqbal, M.M. (2013), "Flexural behavior of cold-formed steel concrete composite beams", *Steel Compos. Struct*., **14**(2), 105-120.
IS 4923-1997, Indian standard hollow steel sections for structural use-specifications, Bureau of Indian Standards, New Delhi.
IS: 1364-2003, Indian standard Hexagon head bolts, screws and nuts of product grade A and B, Bureau of Indian Standards, New Delhi.
IS: 456-2000, Indian standard plain and reinforced concrete code of practice, Bureau of Indian Standards, New Delhi.
Kang, J.Y., Choi, E.S., Chin, W.J. and Lee, J.W. (2007), "Flexural behaviour of concrete-filled steel tube members and its application steel structures", *Int. J. Steel Struct*., **7**, 319-324.
Kaushik K., Sharma A.K. and Kumar, R. (2013), "Modeling and FE analysis of column to beam end-plate bolted connection", *Eng. Solid Mech*., **2**, 51-66
Kim, J., Yoon, J.C. and Kang, B.S. (2007), "Finite element analysis and modeling of structure with bolted joints", *Appl. Math. Model*., **31**(5), 895-911
Knowles, R.B. and Park, R. (1969), "Strength of concrete filled steel tubular columns", *J. Struct. Division – ASCE*, **105**(12), 2565-2587.
Kumari, B., Sharma, S., Sharma, S. and Kwatra, N. (2017), "Monitoring degradation in concrete filled tubular sections using guided waves", *Smart Struct. Syst*., **19**(4), 371-382.
Liu, D., Gho, W.M. and Yuan, J. (2003), "Ultimate capacity of high-strength rectangular concrete-filled steel hollow section stub columns", *J. Constr.Steel Res*., **59**(12), 1499-1515.
Pirmoz, A., Daryan, A.S., Mazaheri, A. and Darbandi, H.E. (2008), "Behavior of bolted angle connections subjected to combined shear force and moment", *J. Constr. Steel Res*., **64**(4), 436-446.
Prabhavathy, A.R. and Samuel KNight, G.M. (2006), "Behaviour of cold-formed steel concrete in filled RH connections and frames", *Steel Compos. Struct*., **6**(1), 71-85.
Schneider, S.P. (1998), "Axially loaded concrete-filled steel tubes", *J. Struct. Eng. - ASCE*, **124**(10), 1125-38.
Shakir, K.H. and Mahmoud, M.A. (1995), "Steel beam connections to concrete-filled tubular columns", Sweden: Nordic steel construction conference, June.
Shams, M. and Saadeghvaziri, M.A. (1997), "State of the art of concrete-filled steel tubular columns", *ACI Struct. J*., **94**(5), 558-571.
Sundarraja, M.C. and Prabhu, G.G. (2013), "Flexural behaviour of CFST members strengthened using CFRP composites", *Steel Compos. Struct*., **15**(6), 623-643.
Tomii, M., Yoshimura, K. and Morishita, Y. (1977), "Experimental studies on concrete filled steel tubular stub columns under concentric loading", *Proceedings of the International Colloquium on Stability of Structures under Static & Dynamic Loads*, Washington, SSRC/ASCE, 718-741.
Uy, B. (2001), "Strength of short concrete filled high strength steel box columns", *J. Constr. Steel Res.*, **57**, 113134.
Wang, J.F. and Guo, S. (2012), "Structural performance of blind bolted end plate joints to concrete-filled thin-walled steel tubular columns", *Thin Wall. Struct*., **60**, 54-68.
Wang, J.F. and Specncer Jr., B.F. (2013), "Experimental and analytical behavior of blind bolted moment connections", *J. Constr. Steel Res*., **82**, 33-47.
Wang, J.F., Han, L.H. and Uy, B. (2009), "Behaviour of flush end plate joints to concrete-filled steel tubular columns", *J. Constr. Steel Res*., **65**, 925-939.
Zhang, J.Q. and Brahmachari, K. (1994), "Flexural behaviour of rectangular tubular sections filled with fibrous high strength concrete", Tubular Structures VI, Rotterdam, Holland, 247-254.
Zhao, X.L. (2002), "Void-filled rectangular hollow section braces subjected to large deformation cyclic axial loading", *J. Struct. Eng*., **128**(6), 746-753.

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