ASGE Vol. 05 (01), pp. 47-56, 2021

Egyptian Knowledge Bank

International Journal of Advances in Structural and Geotechnical Engineering

https://asge.journals.ekb.eg/

Print ISSN 2785-9509 Online ISSN 2812-5142

SEISMIC PERFORMANCE OF STEEL BEAM CONNECTED TO CORNER OF SQUARE CONCRETE FILLED STEEL TUBE COLUMNS

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ABSTRACT

Concrete-filled steel tube (CFST) beam-column joints are susceptible to brittle premature failure due to concentrated local stresses at the tips of the beam tension flange. In order to reduce the outward deformation of the steel tube wall, this paper addresses the influence of setting the steel beam at the corner of the CFST column on the connection performance. Therefore, two specimens represented the details of the common connection and the new proposed connection of exterior joint configuration were sub-assembled and examined experimentally under lateral monotonic loading. Compared with the flexible initial stiffness attained by the common connection, the results showed that the initial stiffness of the proposed connection was relatively rigid up to yielding of the middle region of the width of the beam compression flange under high bearing stresses. The connection strength also increased up to the expected design strength of the connecting beam, while the common connection strength was close to half of the beam design strength. Moreover, after the strength of the beam connection was achieved and at a total rotation of 0.025rad, the failure of the proposed connection was a significant buckling of the beam compression flange propagated to its web and associated with section warping.

Keywords: CFST, Corner, Experiment, Beam-column joint, Welded, Square, Rigid.

INTRODUCTION

The combination of compressive strength of concrete and high ductility of structural steel in the concrete filled steel tube (CFST) column can be more attractive, because of its distinct behavior relative to the traditional structural steel or reinforced concrete columns counterparts. Generally, in addition to canceling the concrete formwork in the construction process, the steel structure around the concrete not only improves the concrete strength capacity and prevents the spalling of the concrete due to its restraint effect, but also minimizes the creep and dry shrinkage of the concrete. At the same time, concrete can delay or prevent the local premature buckling of steel tube, improve the stiffness and strength, and thus reduce the interstory drift demand of the structure. Therefore, the reasonable application of concrete-filled steel tube columns in high-rise buildings has aroused people's attention, but considering the advantages of the CFST, it is not feasible in moment resisting frames with poor connections' seismic performance. In other words, to give full play to the advantages of concrete-filled steel tube, the key is to adopt a satisfactory beam column connection. Therefore, several suggestions of welded and bolted connections are studied to ensure the proper seismic performance of MRFs.

Considering a premature brittle failure modes occurring in the full penetration butt welds which directly attached the beam flange to CFST column, Ansourian, 1976 [1] proved that the poor performance of directly welded connections to the skin of CFST can be improved by transferring the tension component of the beam bending moment to the back wall of CFST column as bearing action. A set of beam to circular CFST connections arranged according to their fabrication difficulty were analytically and experimentally examined by (Alostaz & Schneider, 1996; Schneider & Alostaz, 1998) [2, 3] to evaluate their seismic response. Although circular CFSTs shape was adopted, large tube wall distortion also had occurred. The outward distortion made the beam flanges, the flanges weld, and the tube wall highly susceptible to fracture. Therefore, weldable deformed bars embedded in the concrete and welded to the beam flange can transfer the flange beam tension load to the concrete, and thus prevent the distortion of the tube wall and minimize the chance of earlier fracture of the connection. The good results obtained by (Schneider & Alostaz, 1998) [3] encouraged (Beutel et al., 2001; Beutel et al., 2002) [4, 5] to widely address this suggestion. In addition, external diaphragms can improve the connection behavior provided that smooth transition of the tensile component of the couple beam bending moment to external diaphragm [2]. A participation of the concrete and tube wall in carrying out the tension load can be verified be allowing the individual beam flanges alone or web to pass through the CFST and welded to the steel tube walls as well as the entire beam. The latter represents an ideal rigid connection [6] with high strength, stiffness and satisfactory deformation capacity and ductility rather than all specimens tested by (Schneider & Alostaz, 1998) [3]. However, the fabrication of the connection relatively quite difficult. It implies that improvement of the connection seismic behavior requires complicated connection details with relatively difficult fabrication works (i.e. increase in the cost of the connection fabrication).

The connection seismic performance can be also improved with strengthening the connection zone through various types of diaphragms, for instance; the through diaphragm [8, 9], the interior diaphragm [10], the combined diaphragm (a combination of the modified inner and through diaphragm) [11], and the mixed diaphragm (a combination of the through and exterior diaphragms). Nevertheless, the difficulty of the fabrication and restricting the concrete infill compaction in the construction process were shortcomings associated with those types of connections. Delivering the beam flange tension force indirectly to the tube webs can relieve the outward deformation of the tube wall. Therefore, to improve the connection behavior, external T-stiffeners were proposed by Chung et al., 2005; Shin et al., 2004 [12, 13] to deliver the beam couple forces to the steel tube webs. The through- bolts/ or rods, the blind bolts with or without welded bar extension can be used to alter the normal bolts in the popular flush or extended end plate connections or T-stub connections [10, 14-21]. Those connection were often classified as semi-rigid connection. Therefore, some attention is to be taken with analysis model of the frames with those types of the connections in comparison with common rigid or pinned connections.

This work aims to propose a simple welded connection detail, in which the beam can be joined to a corner of the CFST column rather than the steel tube flange, as seen in Fig.1 (a). The proposed connection is characterized by: simple fabrication without any additional cost, the connection can be shop-welded and field-bolted to ensure high quality of welds as well as it is a two way connection [see Fig.1 (b)]. In addition, it can decrease the beam depth and steel volume, and reduce flared flange beam width (if necessary). From the view point of the structural response, it can increase the structural stiffness, strength, and ductility, and ensure ductile behavior of the weld. Furthermore, restraining the outward deformation makes the beam flanges, steel tube, and flange welds invulnerable to premature brittle failure mode.

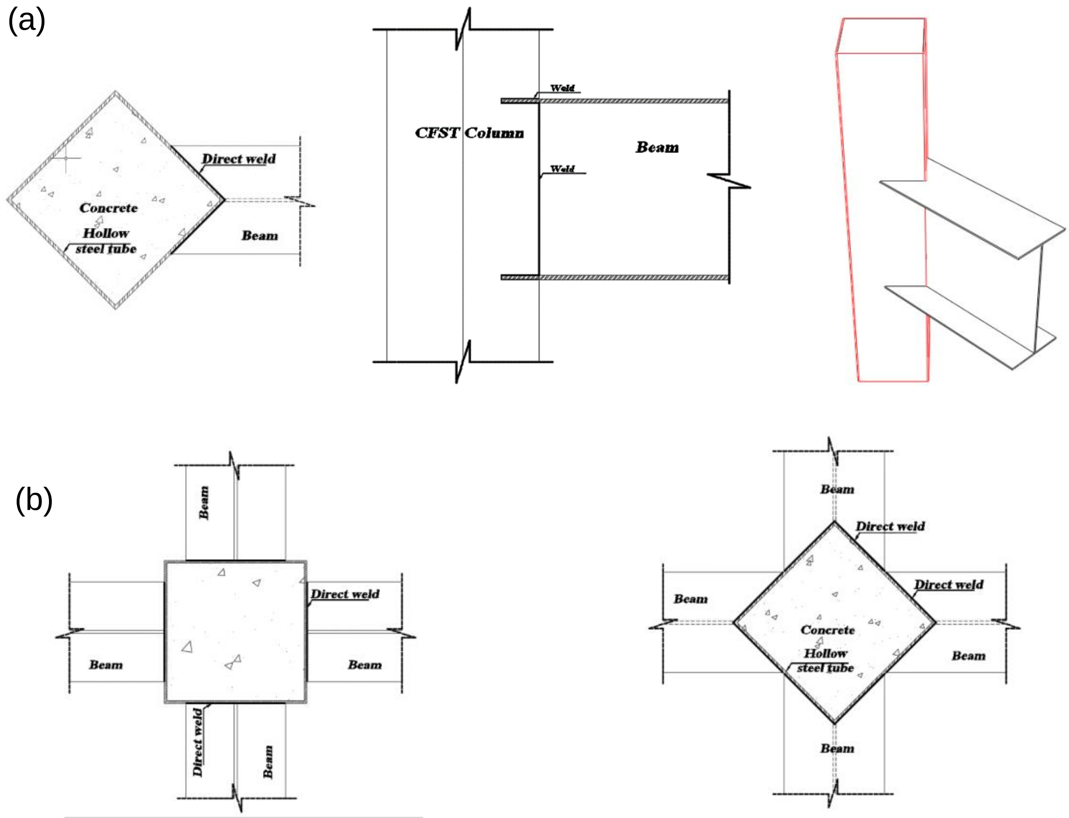

Fig.1: Connection configuration, (a) (one-way) exterior new connection configuration details, (b) two way configuration details of the common and new connections.

EXPERIMENTAL PROCEDURE

Test Setup and Instrumentation

Two specimens representing the beam to CFST column connection were experimentally examined under quasi-static monotonic load up to their failure modes. One of the specimen represents the simple conventional welded connection, in which the beam was welded to the flange wall of the steel tube of the CFST. The other, considering the new joint configuration details [see Fig.1 (a)], was also welded to the steel tube but on its corner. T-shaped beam to column connection representing an exterior plane frame was sub-assembled. It comprised two built up I-beam sections with 70mm flange width, 160mm web depth, 2mm thickness of both flanges and web, and 1000mm length; considering the connections details, each beam was connected to a square hollow steel tubes with 100mm width, 1.9mm thickness and 1000mm height at the mid-height. The steel tube was filled with a concrete as a composite CFST column. The mean strength of the concrete was equal to 38MPa, whereas the average yield stress, ultimate tensile strength and elongation ratio of the built up beam steel plate was 273MPa, 377MPa and 15%, respectively. The steel tube material had 238MPa, 410MPa, and 21% for the yield stress, the ultimate tensile strength, and the percentage of elongation, respectively.
After T-shaped joint configuration of beam to CFST column was completed. The column was placed under 500 ton axial machine and subjected to about 9.8 to 10 ton as axial compressive load. Roller support details were established at the top of the column so that the axial load can be applied, while the support at the bottom of the column can be considered as pinned support when the bolts nuts were loose [see the Photograph in Fig 2 (a)]. The beam was subjected to monotonic load applied at the tip of the beam. Exactly, the distance between the center line of the jack and the outmost fiber of the steel tube was 800mm for both specimens. The beam was supported laterally to prevent any torsional buckling, as shown in Fig.2 (a).

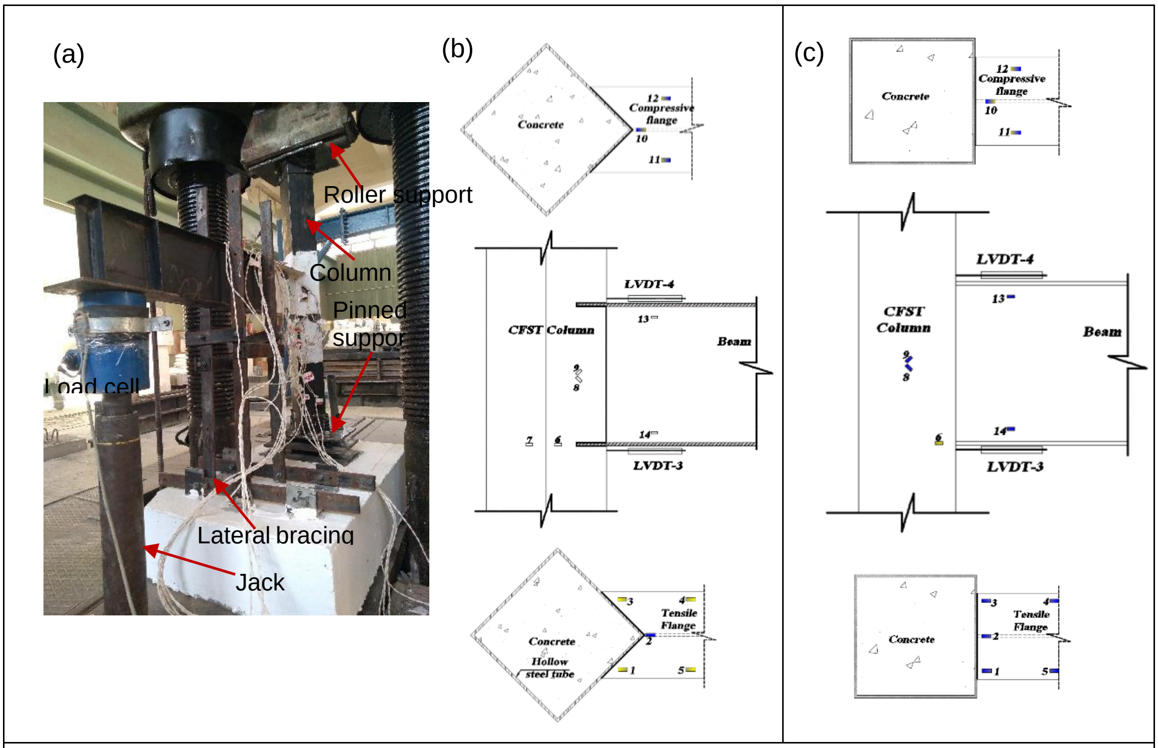

Fig. 2: Test setup and instrumentations details, (a) test setup photograph, (b) instrumentation of the new connection, (b) instrumentation of the common connection

Instrumentation

Figures 2 (b & c) illustrate the arrangement of the strain gauges on both connections components (beams and columns). The strain gauges (1 to 5) monitored the tensile beam flange to describe the strain distribution in the vicinity of the connection, whereas the strain gauges (10-12) were placed on the compression flange to describe the compressive strain distribution. Two strain gauges were set on the beam depth (13&14), and (6&7) was put on the column face align with the line of the beam tension flange but on the different orthogonal steel tube walls. Another two strain perpendicular to each other (8&9) rotating by 45 about the normal axis of the steel tube wall were set to study the panel zone shear strain. Four load vertical displacement transducers (LVDTs) were arrangement, so that LVDTs (1&2) were used to measure the vertical displacement of the beam in two predetermined points (at the load point position and at the distance of 210mm from the face of the CFST column). The LVDTs (3&4) near the outside of the beam flanges were to measure the contribution of the column in total rotation of the connection.

EXPERIMENTAL RESULTS AND DISCUSSION

Failure Modes

Common Connection

Regarding the common connection, during the loading process, earlier fracture of the steel tube wall at the tensile beam flange region was observed (exactly, at one tip of the beam flange). At the load of 2kN, clear sound was heard, and then, at about 3.5kN, an evident crack initiated on the steel tube wall around one edge of the tensile beam flange. The crack developed to be apparent tear at the edge of the beam flange. Up to 6kN, the crack propagated along the outside of the tensile beam flange with evident outward deformation of the tube wall. The connection showed inability to sustain more loading than 6kN. At the same time, the crack developed to obvious tearing along the outside of the beam tensile flange (with clear sound of the steel plate tearing) and toward compression flange with a noticeable decreasing of the strength. Upon successive loading, the tear was extended and the load capacity showed a continuous decrease. The beam and concrete material seemed sound at the end of the loading. Figure 3 (a) depicts the failure mode of the common connection.

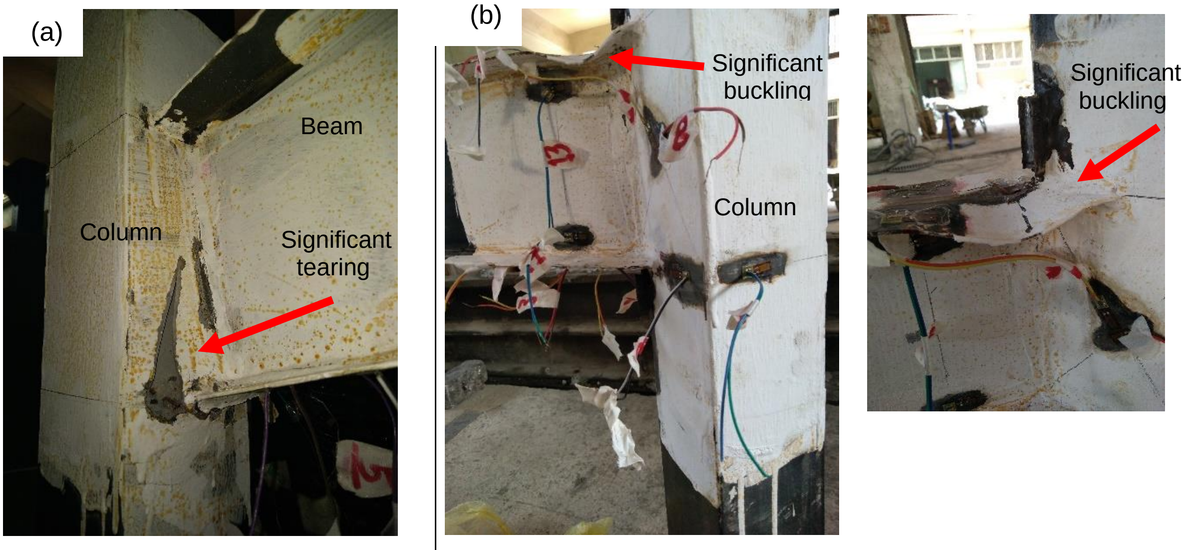

Fig. 3: Failure modes at the end of experiments, (a) common connection, (b) new connection

New Connection

Figure 3 (b) shows obviously the failure mode of the new connection at the end of the loading process. Compared with the common connection, an emission of sound associated with 11kN to 11.5kN was heard. At the same time, an evident initial buckling on the beam compression flange was developed to be obvious. Comply with 12.5kN, the beam flange buckling became significant, but it concentrated on one side of the beam compressive flange associated with beam web buckling [see the photograph in Fig.3 (b)] and clear warping of the beam section at the applied load point. It is worth noting that earlier significant local yield was developed on the mid-width of the beam compressive flange at the corner of the steel tube, due to concentrated local bearing stress [please, refer to the strain state of the strain gauge 10 in Fig.6]. Consequently, compared with the sensitivity of steel tube wall, flange weld and beam tension flange of the common connection to premature failure under tension couple of the beam bending moment, the new connection is more prone to beam flange and web buckling under compression couple of the beam bending moment. Hollow steel tube and concrete material were intact with insignificant observed deformation at the end of loading process.

Connections Performance

The force-displacement curve and moment-rotation relationship in Fig.4 (a & b) demonstrate that when the beam was connected to the CFST column at the corner, corresponding to previously proposed details, the connection response was perfectly enhanced in term of strength and stiffness compared with traditional directly welded connection counterpart (joining the beam exclusively to the steel tube flange only). To be exact, the strength of the new connection was increased by about twice as much as that of the traditional connection before the strength decreased significantly due to local buckling of the compression flange of the beam. That is, the expected design strength of the beam was achieved by new connection before the beam exposed a significant local flange buckling, whereas the common connection closed up half strength capacity of the beam before tearing the column steel tube wall. The initial stiffness was rigid, but due to excessive local bearing stress at the mid-width of the beam, compression flange constituted significant local yielding at the mid-width of the beam flange causing an obvious decrease in the rigidity of the connection. However, the stiffness was still greater than the stiffness capacity of the common connection by about twice. That can be attributed to redistribution of the compressive stress over the compression beam flange and web zone (geometric and metallic strain hardening). The rotation capacity of the new connection was insufficient where the total rotation of the connection was about 0.025rad, and this can be attributed to that, the flange of the beam was non-compact as well as the significant local bearing stress on the compression beam flange. The deformation capacity of the traditional connection is significantly high but it associated with fracture of the tube wall and the connection can be classified as pinned connection.

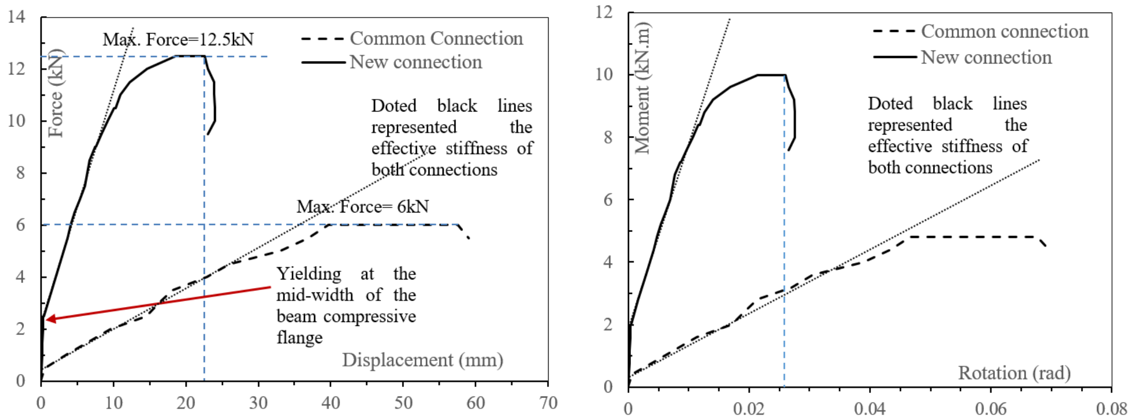

Fig. 4 : Response of both connections, (a) Force-displacement relationship, (b) Moment-rotation relationship

Strain States

Due to premature failure of the common connection by excessive fracture of the steel tube wall align with the beam tension flange level, the strain state of the overall strain gauges did not exceed the yielding strain of their steel material. Therefore, this section focused primarily on the discussion and the evaluation of the strain state of the new connection's components. Considering the beam tension flange, Fig.5 (a) shows that the tensile strength at the mid-width of the tensile flange is greater than that at the flange tips. This behavior agrees with the behavior of the tensile flange of a beam welded directly to the flange of the structural wide flange column [22]. Therefore, only the strain at this position significantly exceeded the yield strain of the steel material. Strain states of the strain gauges (4 and 5) at a distance 60mm from the corner of column indicated that the strain and thus the stress are approximately even distributed along the tensile beam flange width. Similarly, as shown in Fig. 6, the compressive strain at the mid-width of the beam compression flange is significantly amplified in earlier stage, so that the strain gauge reached the compressive strain yielding of the material at the load of 2.5kN. Following that, the compressive strain altered toward tension strain as an indicator for the initial local bulging. Excessive increase in the tensile strain at this region can trigger the beam flange to buckle upon continuous loading process. On the contrary, the strains at the tip of the compression flange at a distance 60mm from the column corner was less than yielding strain of the material prior to altering the compressive strain toward tensile strain, as an indicator to the onset of flange local buckling, as observed in Fig.7 (a). The earlier and rapid buckling might be attributed to the effect of the non-compact section of the beam flange besides the concentrated local bearing stresses at the mid-width of the compressive flange. Considering the beam web, prior to the beam web buckling, the compressive strain of the beam web near the compressive flange roughly good agreed with its tensile strain near the tensile flange, as shown in Fig.8 (a). The strain state of the beam section at 60mm far from the corner of column (strain gauges. 4, 5, 11, 12, 13, 14) indicate that the beam buckled in elastic stage because overall strains at the beam section did not exceed the elastic strain of the material. This is reasonable result due to non-compact flanges. However the strains were close to material yielding strain. Regarding the steel tube, Fig. 9 (a) exhibits that, the compressive strain in the panel zone did not exceed the yielding strain (Straingauage09 in the Fig. 9 (a)), whilst the tensile strain in the panel zone and the tensile strain on the steel tube align with the beam tension flange approached closely from the material tensile strain. The back walls of the steel tube parallel to walls connecting to beam flanges sustained insignificant tensile load according to the tensile strain of straingauge07 in Fig. 9 (a), which was set align with the beam tension flange but at the back wall of the steel tube [refer to Fig.2 (b)]. Considering the strain state of the members of common connection, as seen in Figs. 5, 7, 8, 9 (b), the material was not exploited due to the occurrence of an earlier fracture on the steel tube wall prior to the connection reaching to the nominal strength of the beam's connection.

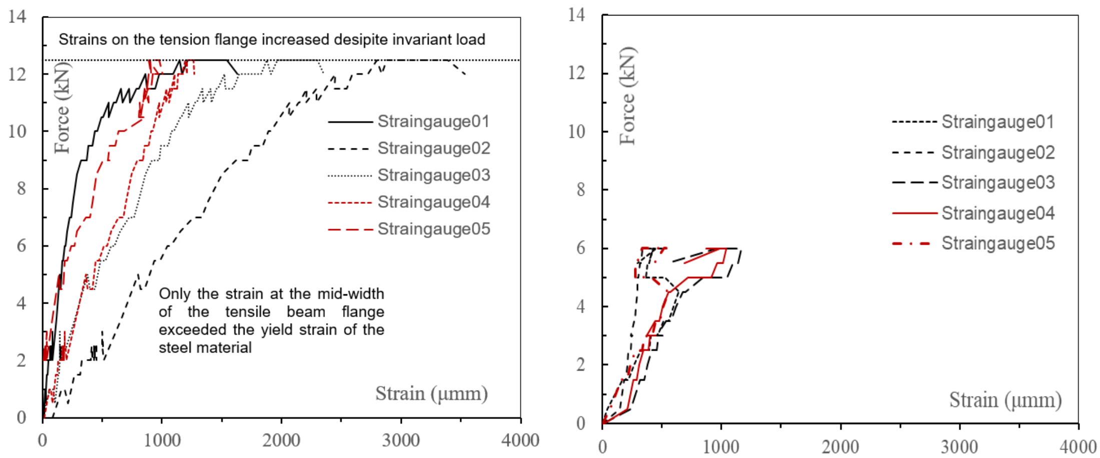

Fig. 5: Strain state of the beam tension flange, (a) new connection, (b) common connection

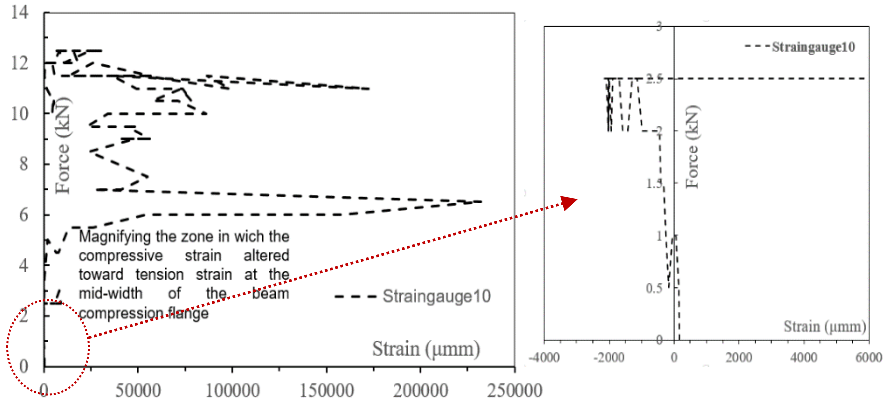

Fig. 6: Strain state at acritical point on the middle width of the beam compression flange

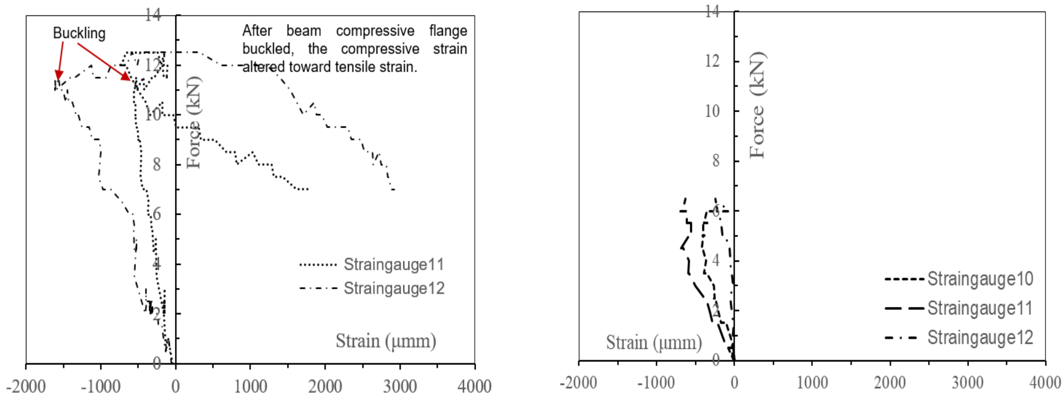

Fig. 7: Strain state of the beam compression flange, (a) new connection, (b) common connection

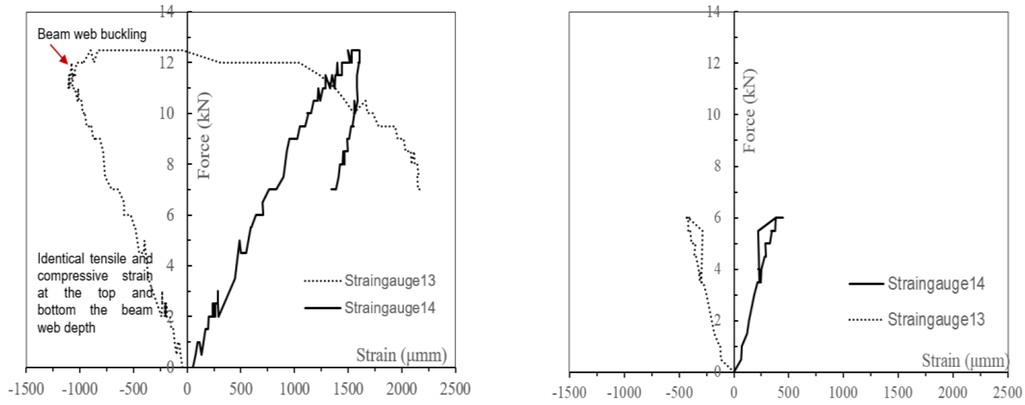

Fig. 8: Strain state of the beam web, (a) new connection, (b) common connection

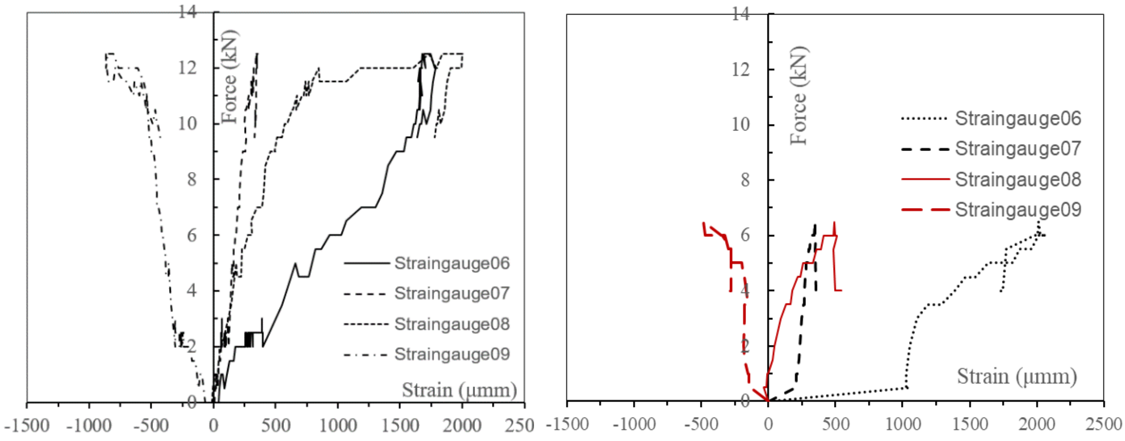

Fig. 9: Strain state of the column steel tube, (a) new connection, (b) common connection

CONCLUSIONS

The purpose of connecting I-beams to the corners of CFST columns is to reduce the high concentrated tensile stress at each tip of the beam tensile flange, which usually occurs in ordinary connections. The test results revealed the main key findings:

1. The proposed details contributed well to delay the failure mode, develop high rigidity, achieve excellent strength capacity, realize acceptable deformation capacity, and provide sufficient ductility of the beam-column CFST connections. On the contrary, the common connection had poor performance in term of the initial stiffness, strength, and ductility. Its high deformation capacity was due to significant wall distortion of the column's steel tube and rigorous tearing of the column steel tube wall.
2. The proposed connection achieved the theoretical beam design strength compared with achieving only 50% of the theoretical value by the common connection.
3. The initial stiffness of the proposed connection can be classified as rigid connection; however, the common connection was close to pinned connection according to applicable seismic design codes.
4. The total rotation of the proposed connection was 0.025rad, which indicates the importance of further research to satisfy the provisions of seismic design codes for special moment resisting frames.

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