BOLTED BEAM-COLUMN CONNECTIONS FOR CONCRETE-FILLED TUBE STRUCTURES

L. L. Chung
National Center for Research on Earthquake Engineering, Taipei, Taiwan
chung@ncree.gov.tw

L. Y. Wu, T. J. Shen, G. L. Huang, and S. F. Tsai
Department of Civil Engineering, National Taiwan University, Taipei, Taiwan

Abstract

In this paper, a bolted connection for rectangular concrete-filled steel tube column and H-shape steel beams is proposed so that no welding is required at site. Moreover, the flanges of the beam close to the column are widen by welding additional triangular plates to both sides of the flanges so that plastic hinges are guaranteed to be developed away from the welding zone. The seismic performance of the connection detail is investigated by subjecting the test specimens to cyclic loading tests. The feasibility of the proposed beam-column connection is successfully verified by experiments. The story drift ratio can be up to 6 % radian without collapse. The failure mechanism depends on the relative strength of the beam, column and panel zone. When the width-to-thickness ratio of the column is large, energy dissipation is mainly attributed to the failure of the panel zone. On the other hand, when the width-to-thickness ratio is small, energy dissipation is mainly attributed to the failure of the beams.

INTRODUCTION

The properties of steel and concrete are fully utilized in the concrete-filled steel tube structures so that the strength, stiffness and ductility of the structures can be enhanced simultaneously. The construction of concrete becomes more efficient because the form-work for concrete is provided by the steel tube. Since longitudinal reinforcement and transverse confinement can be acquired due to presence of the steel tubes, the traditional longitudinal and transverse reinforcement may be eliminated. Continuous confinement provided by the steel tubes prevents excessive spalling of concrete and the concrete filled inside the steel tubes prohibits local inward buckling of the steel tube wall. Even though many advantages are exhibited by the concrete-filled steel tube structures, the application is limited because of the complicated beam-column connection and the lack of construction experience (AIJ 1987; Alostaz and Schneider 1996; Ricles et al. 1997).

In this paper, three full-scale connections, each composed of a rectangular concrete-filled steel tube column and a pair of H-shape steel beams, were subjected to cyclic loading tests, so that the seismic performance of the connection detail can be investigated. With bolted connection, welding is eliminated at site so that the quality of welding is no longer a problem. H-shape steel beam is welded with end plate at shop and holes are allocated on the end plate. Holes are also allocated on the rectangular steel tube with the same pattern. After concrete is placed in the tube and its strength is developed, the bolts are pre-stressed. Moreover, the flanges of the beam close to the end plates is widen by welding additional triangular plates to both sides of the flanges so that plastic hinges are guaranteed to be developed away from the welding zone.

DETAIL OF BEAM-COLUMN CONNECTIONS

The test specimens are constructed in the cruciform shape to simulate the full-scale interior sub-structure of a building with span 6 m and story height 3.2 m (Fig. 1). The columns are square steel tubes with cross-section 400×400 mm and the thickness of the tubes varies from 6, 8 to 10 mm. Meanwhile, the beams are commercially available H-shape steel with cross-section 500×200×10×16 mm for all test specimens. Since the square tubes with the required size are not available in the market, the tubes are built up from steel plates. The steel plates are first cold-bent into U-shape and two U-shape channels are jointed together by complete penetration groove welds with backing strip to form a square tube. There are totally thirty-two pipe sleeves, sixteen in each direction, embedded in the panel zone of the square tube. An end plate with thickness 25 mm is jointed to the end of the beam by complete penetration groove welds. Holes, conformable with the pipe sleeves, are allocated on the end plate. Two beams are connected to the column by bolting tie rods through the pipe sleeves in the panel zone and the holes on the end plates. In order to shift the plastic zone away from the column face, triangular plates are appended to the flange of the beams by complete penetration groove welds. Moreover, the end of the beam is further strengthened by welding triangular plates, which are aligned with the web of the beam, vertical to the flanges and the end plate.

The test specimens are named as FSB6, FSB8 and FSB10 where F denotes that the steel tube column is filled with concrete, S denotes square tube, B denotes bolted connection and the number denotes the thickness of the square tube in millimeters.

EXPERIMENTAL SETUP

The specimen in the cruciform shape represents two consecutive stories and two consecutive spans (Fig. 2). The top of the column held by a horizontal actuator represents the point of inflection of the upper story while the bottom of the column held by a short H-shape beam represents the point of inflection of the lower story. The left and right ends of the beam held by vertical actuators represent the points of inflection of the left and right spans, respectively. A cross beam is mounted on the top of the column in the out-of-plane direction. A pair of tie rods, bolted to the cross beam and the strong floor, are pre-stressed by oil jack so that dead load and live load are simulated. In order to minimize the material used and simplify the procedure of specimen installation, the pair of beams are extended to the required length by a pair of transferred beams. Since the transferred beams remain elastic during the test, they can be used repeatedly. The horizontal actuator and the two vertical actuators are all operated in the displacement control mode. The horizontal actuator holds the top of the column in position with zero displacement while the two actuators move synchronously in opposite direction. The cyclic story drift produced by the two vertical actuators simulates the seismic load.

EXPERIMENTAL RESULTS

The forces and displacements of the vertical actuators applied to the points of inflection of the beam for all specimens are shown in Figs. 3-5. The failure mechanisms of the connections are shown in Figs. 6-8.

In the specimen FSB6, the thickness of the column tube was 6 mm and concrete was filled inside the tube. When the displacement of the beams was less than 60 mm (2 % radian), only the whitewash of the triangular plates vertical to the flanges and the end plates of the beams spalled slightly. When the displacement of the beams was 60 mm (2 % radian), the whitewash cracked horizontally between the holes in the panel zone and the column web in the panel zone bulged out slightly. The whitewash on the flanges of the beams also spalled slightly. When the displacement of the beams was 90 mm (3 % radian), the whitewash in the panel zone cracked horizontally and vertically, and the prying action of the end plates was observed. When the displacement of the beams was 120 mm (4 % radian), the column web in the panel zone bulged out and the holes became ellipse. The concrete in the panel zone lost its function as confinement and cracked along the holes. When the displacement of the beams was 150 mm (5 % radian), the strength of the connection reached its maximum value. The column web in the panel zone buckled outward and the column flanges above the end plates also buckled locally. When the displacement of the beams was 180 mm (6 % radian), stress and strain were concentrated around the holes in the panel zone. The column web in the panel zone buckled seriously and the concrete in the panel zone crushed. When the displacement of the beams was 210 mm (7 % radian), the column web in the panel zone cracked from the hole but the strength of the connection had not been attenuated to 80 % of its maximum value yet.

In the specimen FSB8, the thickness of the column tube was 8 mm and the tube was filled with concrete. When the displacement of the beams was less than 30 mm (1 % radian), only the whitewash of the triangular plates vertical to the flanges and the end plates of the beams spalled slightly. When the displacement of the beams was 30 mm (1 % radian), the whitewash on the flanges of the beams began spalling. When the displacement of the beams was 45 mm (1.5 % radian), the whitewash in the panel zone cracked horizontally and vertically between the holes. When the displacement of the beams was 90 mm (3 % radian), the whitewash kept on cracking and spalling. When the displacement of the beams was 120 mm (4 % radian), the flanges of the beams buckled slightly and the prying action was observed in the end plates. When the displacement of the beams was 150 mm (5 % radian), the strength of the connection reached its maximum value. The deformation of the column web in the panel zone concentrated around the holes and the buckling of the beam flanges became significant. Moreover, the end plates were observed to begin deforming. When the displacement of the beams was 160 mm (6 % radian), the holes in the panel zone deformed to be ellipse and the column web in the panel zone buckled locally. When the displacement of the beams was 210 mm (7 % radian), the column web fractured from the holes in the panel zone.

In the specimen FSB10, the thickness of the column tube was 10 mm and the tube was filled with concrete. When the displacement of the beams was less than 30 mm (1 % radian), only the whitewash of the triangular plates vertical to the flanges and the end plates of the beams spalled slightly. When the displacement of the beams was 30 mm (1 % radian), the whitewash on the flanges of the beams began spalling. When the displacement of the beams was 45 mm (1.5 % radian), the whitewash in the panel zone cracked horizontally and the whitewash of the beam webs also cracked due to flexure. When the displacement of the beams was 90 mm (3 % radian), the whitewash in the panel zone kept on cracking horizontally and vertically between the holes. When the displacement of the beams was 120 mm (4 % radian), the buckling tendency and prying action were respectively observed from the flanges and end plates of the beams. When the displacement of the beams was 150 mm (5 % radian), the strength of the connection reached its maximum value. The beam flanges buckled seriously. When the displacement of the beams was 180 mm (6 % radian), the beam flanges buckled more seriously but the concrete in the panel zone only cracked along the holes.

During the cyclic tests, the connections dissipated the energy through the columns, beams and panel zones (Fig. 9). In the specimen FSB6, the column tube buckled locally, the column webs in the panel zone cracked between the holes and the concrete in the panel zone crushed. Therefore, energy dissipation was mainly attributed to the failure of the panel zone. In the specimen FSB8, the beam flanges buckled locally, and the column webs and the concrete in the panel zone cracked between the holes. Therefore, energy dissipation was attributed to both the failure of the beam and the panel zone. In the specimen FSB10, both the flanges and the webs of the beams buckled locally. Therefore, energy dissipation was mainly attributed to the failure of the beams.

CONCLUSIONS

The feasibility of bolted connection for concrete-filled steel tube and steel beam is successfully verified by cyclic testing. The story drift ratio can be up to 6 % radian without collapse so that the seismic performance of the proposed connection is satisfactory. The failure mechanism depends on the relative strength of the beam, column and panel zone. When the width-to-thickness ratio of the column tube is large, energy dissipation is mainly attributed to the failure of the panel zone. On the other hand, when the width-to-thickness ratio of the column tube is small, energy dissipation is mainly attributed to the failure of the beams. Since both sides of the flanges close to the end plate are appended with triangular plates and the connection is furthered stiffened by welding triangular plates vertical to the flanges and the end plate, the plastic hinge is successfully shifted away from the welding zone and no brittle failure of the welding is observed.

ACKNOWLEDGEMENTS

This research was supported by the National Science Council and the National Center for Research on Earthquake Engineering. This support is greatly appreciated.

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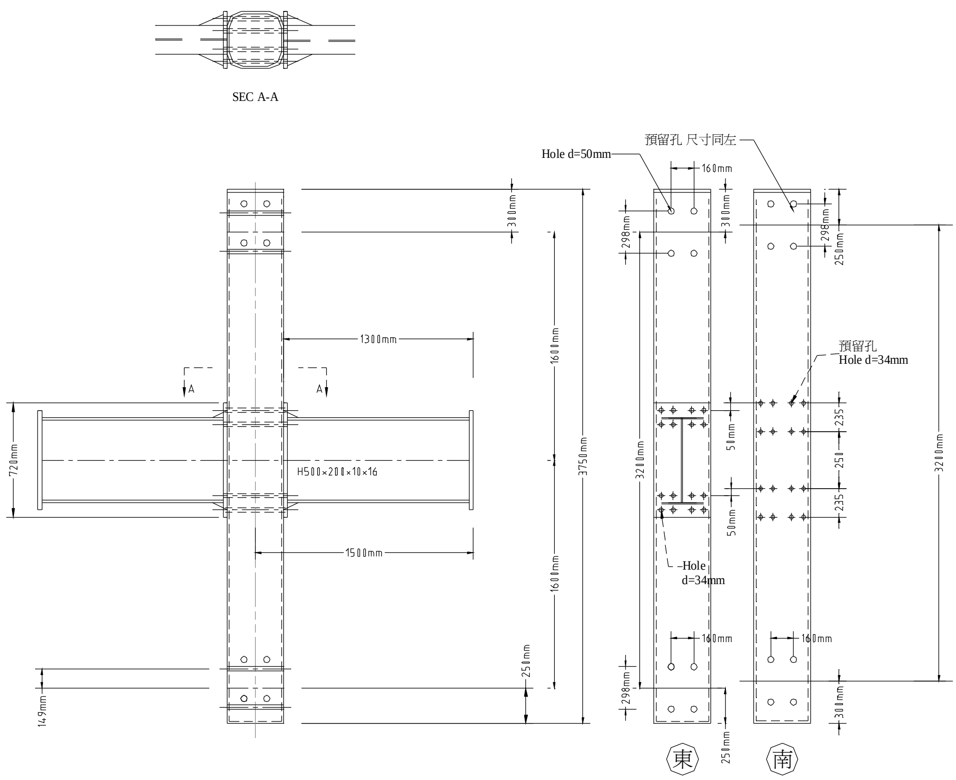

Fig. 1 Design detail for beam-column connections

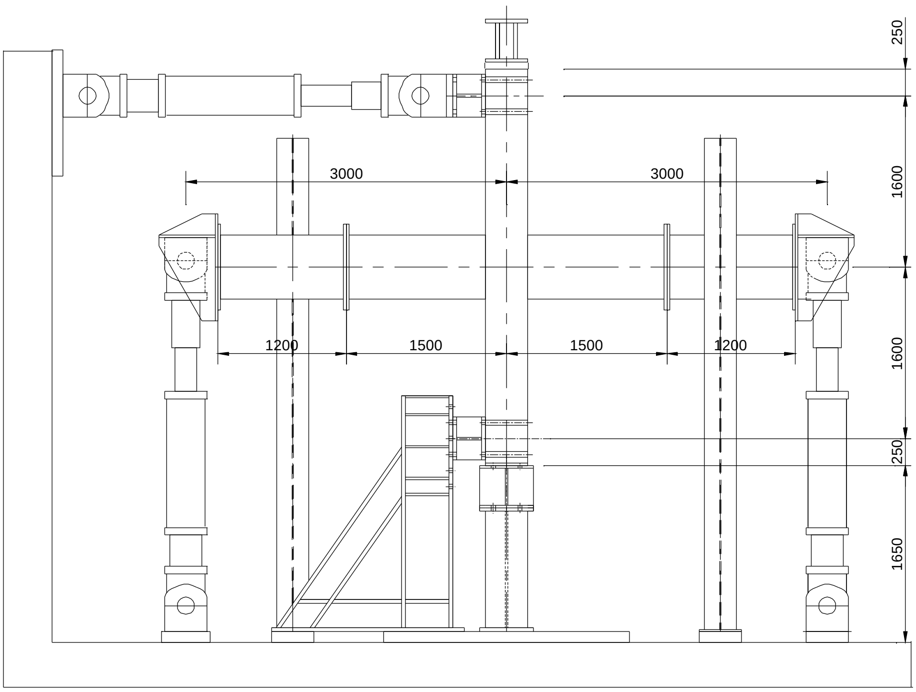

Fig. 2 Experimental Setup

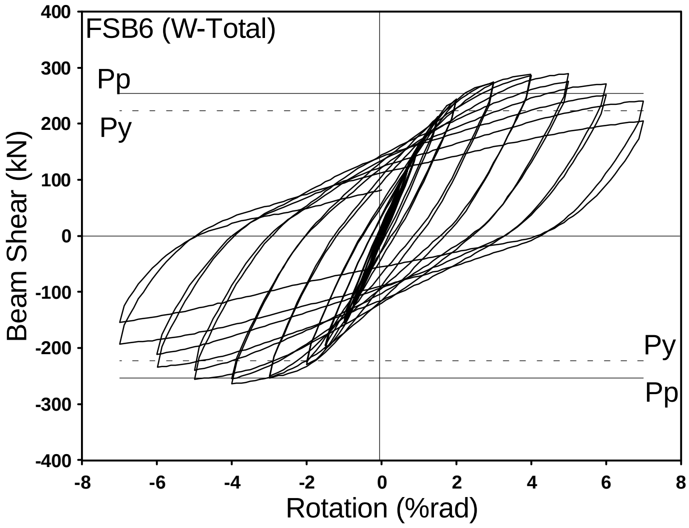

Fig. 3 Force and displacement for FSB6

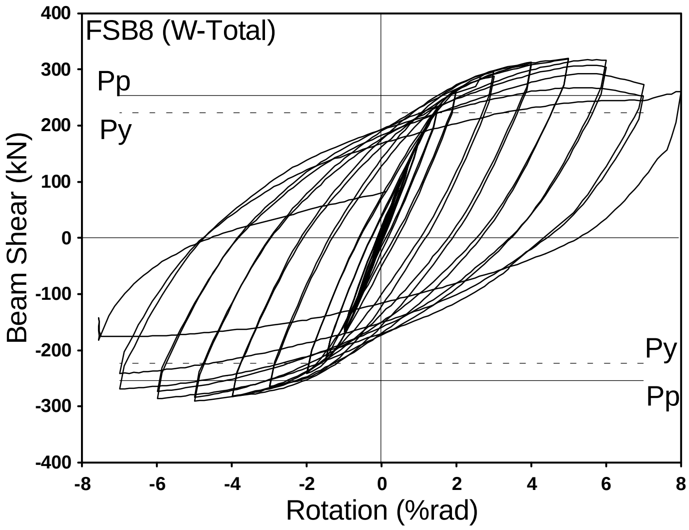

Fig. 4 Force and displacement for FSB8

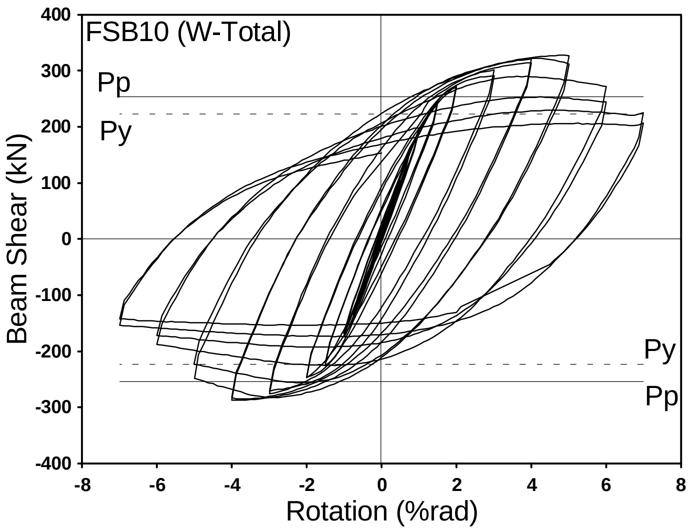

Fig. 5 Force and displacement for FSB10

Fig. 6 Failure of FSB6

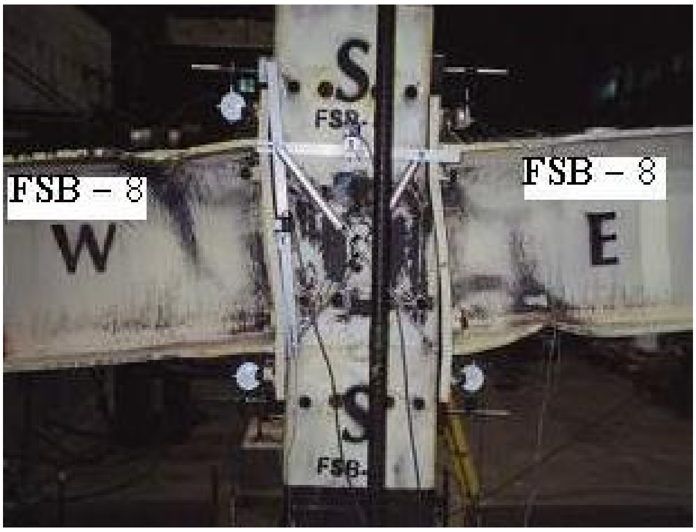

Fig. 7 Failure of FSB8

Fig. 8 Failure of FSB10

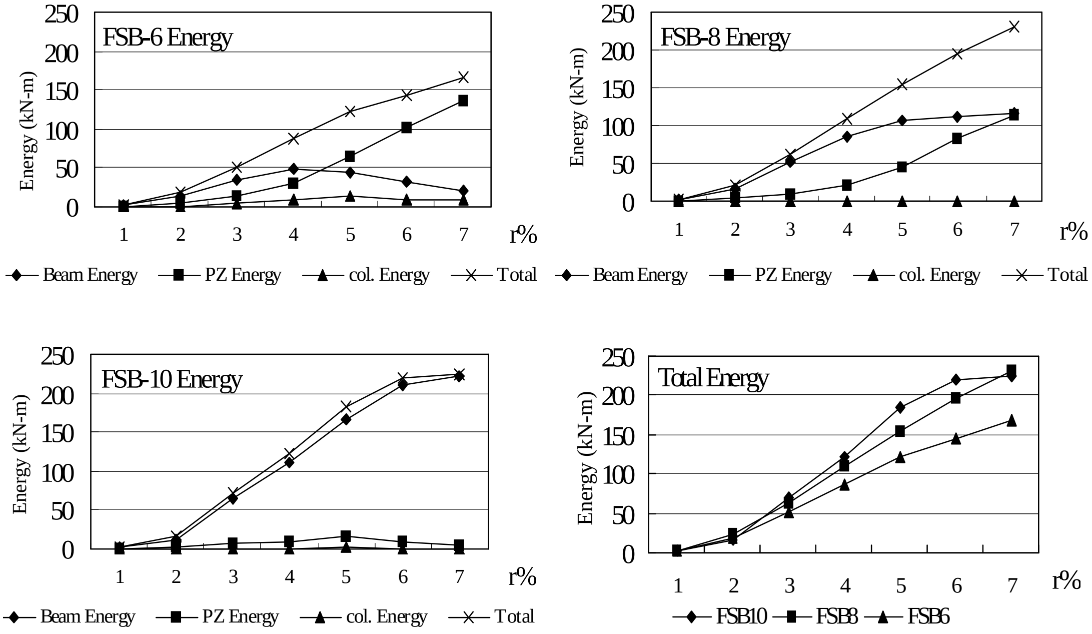

Fig. 9 Energy dissipation for FSB6, FSB8 and FSB10