*17th World Conference on Earthquake Engineering, 17WCEE*
*Sendai, Japan - September 13th to 18th 2020*

# STATIC AND DYNAMIC TEST OF FULLY-BOLTED CONNECTION WITH CORE TUBE FOR CLOSED-SECTION COLUMN IN STEEL STRUCTURE

Yanxia Zhang(1), Mengyao Cheng(2), Binglong Wu(3), Zhanyang Pang(4)

(1) *Professor, Beijing University of Civil Engineering and Architecture, Beijing, China, zhangyanxia@bucea.edu.cn*
(2)*Postgraduate, Beijing University of Civil Engineering and Architecture, Beijing, China, 827138256@qq.com*
(3)*Postgraduate, Beijing University of Civil Engineering and Architecture, Beijing, China, buceawbl@163.com*
(4)*Postgraduate, Beijing University of Civil Engineering and Architecture, Beijing, China, 15731116944@126.com*

## *Abstract*

A new fully-bolted column connection with core tube for closed-section columns in steel structures is proposed in this paper, quasi-static reciprocating tests towards a series of column connection specimens with various parameters have been completed, and dynamic shaking table tests on two 1/4 scale, 5-storey spatial steel frames adopting fully-bolted column connections with and without core tubes have also been conducted. It was indicated by the static test that the adoption of core tube increased the bearing capacity of the column connection by 2%~14%, reduced the bolts tension by 24.7%~46.4%, and greatly enhanced the hysteretic performance and energy consumption capacity of the connections. Shaking table test results revealed that the new fully-bolted connections without core tubes lost rigid characteristic under extremely rare earthquakes; while connections with core tubes showed typical characteristics of the rigid connection. It can be concluded that the new fully-bolted connection with core tube can achieve identical rigid connection to that of the traditional welded connection and are suitable for the multi-rise and high-rise steel structural buildings located at high-intensity earthquake regions.

*Keywords: static and dynamic test; fully-bolted connection; core tube; closed-section column; steel structure*

## 1. Introduction

Box columns are widely used in steel structures due to their equal bending stiffness along the two main axes. In view of the potential issues existed in traditional welded steel column connections such as low construction efficiency, high labor costs, poor seismic performance and severe environmental pollution et al, scholars at home and abroad have conducted researches on the development of new technologies for efficient connection of steel structures and the realization of fully-bolted connections. Sumner et al.[1] proposed an end plate-type beam-column connection and proved that it could be used in beam-column seismic connection, and the ultimate strength of the connection should be greater than the beam strength. J. Lee et al.[2] proposed three types of bolted box column to beam connections through T-pieces, channel steel connectors and bolts, and carried out tests, numerical simulations and theoretical analysis. Zhang et al.[3~7] proposed a fully-bolted truss beam-column jointwhich was validated to have a good plastic rotation capacity, and the theoretical value of the strength of the bolt connection was in good agreement with the experimental value; Li et al.[8] carried out tests and finite element analysis on a concrete-filled steel tube column-column connection of an upper column and a lower column through an outer plate and a self-locking blind bolt and the results showed that the node had sufficient strength. To sum up, the researches on the fully-bolted and high-efficiency connections of steel structures were mainly focused on the beam-column connection. Fewer studies were found to concentrate on the fully-bolted column-column connections of closed section columns, for which most of them were . Based on the previous researches [9-11], A new fully-bolted column connection with core tube for closed-section columns was proposd in this paper. Quasi-static tests on two kinds of the fully-bolted box-column connections with core tube and a welded connection were carried out to study the hysteretic performance, ductility, strain variation and energy consumption performance of the fully-bolted box-column connection. And the mechanical properties of the new fully-bolted connections and the welded connection were compared. At the same time, the dynamic shaking table tests of two steel frame models adopting fully-bolted connections with and without core tubes were conducted. Comparative studies on the failure modes of the both steel frame models under simulated ground motions with various intensities were described to explain the seismic performances of both steel frames.

## 2. Test of the fully-bolted column connection containing core tube

### 2.1 Connection Details

The configuration of the fully-bolted column connection with core tube is shown in Fig. 1. The upper and lower columns are connected by high-strength bolts through upper and lower flanges, where the lower column flange is set to be flush with the upper beam flange. A drive-through clapboard is installed at the postion of the lower beam flange, and its center line should be aligned with the center line of the lower beam flange. The part between the clapboard and the lower column flange is a middle column seat. In order to improve the mechanical performance and strengthen the core area of the column connection, an octagonal core tube is set to enhance the connection of the upper and lower columns. The core tube is welded with the middle column base in the factory as a whole, and then welded with the lower flange plate and the lower column as a whole. Details are shown in Fig.2. After that, the upper and lower columns are connected by high-strength bolts at the construction site. To ensure the cooperative work of the core tube and the column, the total gap between the core tube and the column wall shall not exceed 2mm.

.

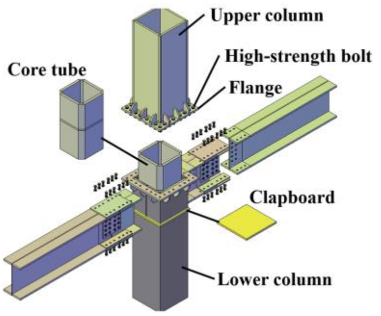



Fig.1 Column connected by core sleeve and bolt

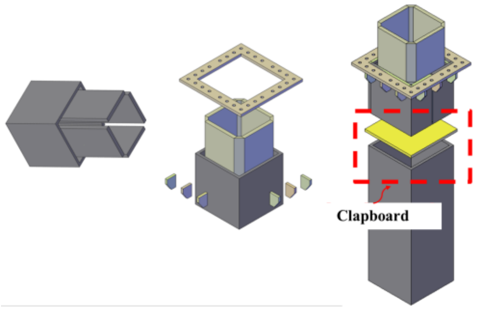



Fig.2 Installation of core tube and lower column

### 2.2 Test design

#### 2.2.1 Specimen design

The main parameters of the three test specimens are listed in Table 1. The fully-bolted connection with cure tube (i.e. CS-1) is set to be the basic test speciman. Comparative specimans include the fully-bolted connection without core tube (i.e. CS-2) and the welded column connection (i.e. WD-1), which were designed to investigate the influence of setting core tube and to study the rigid performance of the new column connection CS-1. The steel material of all specimens is Q345. The total height of the column is 2170mm, where the upper column is 1345mm and the lower portion is 825mm. The cross section of the box column is 300mm*300mm*16mm*16mm. Diaphragms are installed in the upper and lower columns. The length and wedth of the octagonal core tube is 360mm and 22mm, respectively. The size of flange plate is 450mm*450mm*18mm. The upper and lower flange plate are connected by twelve M20, 10.9s hign-strengh bolts. The detailed sizes of the specimens are shown in Fig.3.

Table 1 Main parameters of specimens

| Specimen number | Connection type | Bolt grade | Core tube |
|---|---|---|---|
| CS-1 | fully-bolted connection with core tube | 10.9s | set |
| CS-2 | fully-bolted connection without core tube | 10.9s | —— |
| WD-1 | welded connection | —— | —— |

w



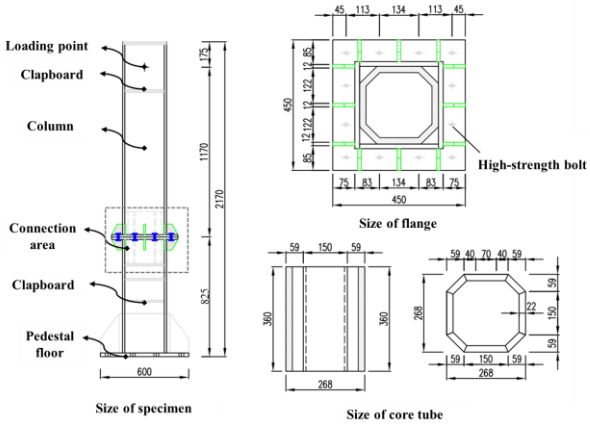



Fig. 3 Detailed sizes of specimens

### 2.2.2 Test loading scheme

Horizontal cyclic loading is applied to the specimen under vertical axial force [12]. The axial force is applied at the top of the column by a 2000kN actuator. A skateboard is installed between the actuator and the reaction frame to ensure that the actuator can slide as the joint deforms. Displacement loading is applied at the inflection point of the specimen using a 2000kN actuator in the horizontal direction. One end of the actuator is fixed at the connector installed at the top of the column, and the other end is connected to the reaction wall. Column foot are connected with the floor beam by twelve M30 bolts to simulate the fixed end. Test loading devices are shown in Fig. 4.

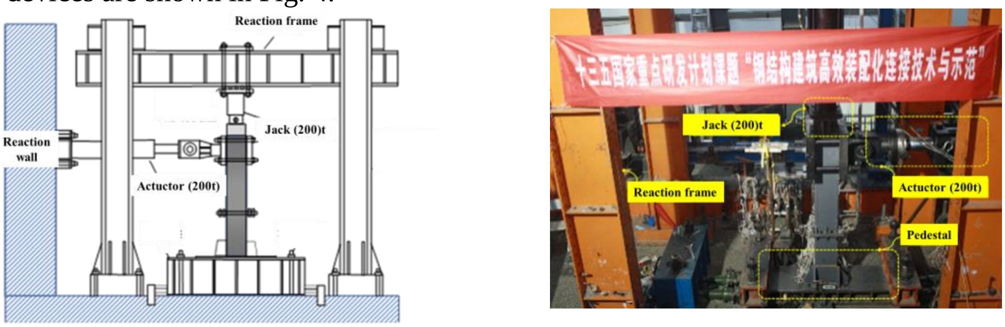



Fig. 4 Test loading devices

During the test, a axial force of 1250 kN was applied at the column top, and the displacement loading scheme was adopted to exert cyclic loading in the lateral side according to the American AISC seismic code [13], where the test loading was performed on the basis of the interstory drift $\theta$. The loading amplitudes of every loading level were 0.00375rad (7.48mm)、0.005rad (9.98mm)、0.0075rad (14.96mm)、0.01rad (19.95mm)、0.02rad (39.9mm)、0.03rad (59.85mm)、0.04rad (79.80mm)、0.05rad (99.75mm) and 0.06rad (119.70mm), respectively and 2 cycles for per loading level was implemented. The test shall be terminated when severe damage of specimens occurs or the load drops to 85% of the peak load.

## 2.3 Test phenomena and results

### 2.3.1 Test phenomena

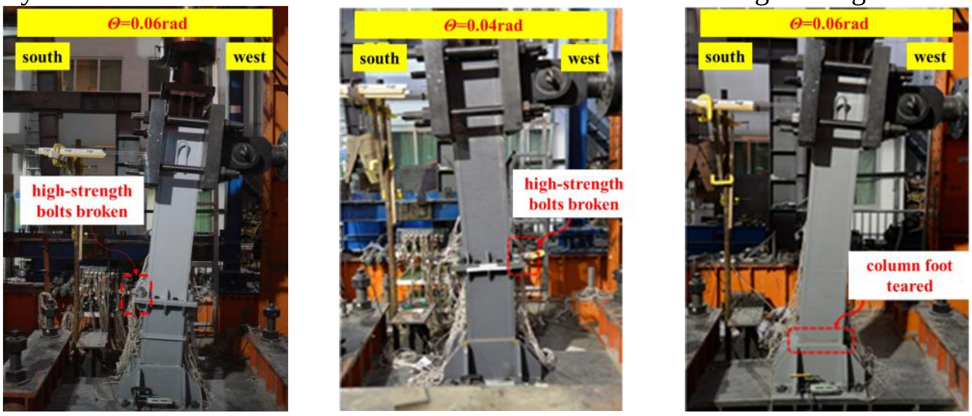
During the entire loading process, each specimen has undergone the elastic stage, elastoplastic stage and plastic deformation stage. Before loading level of 0.02 rad, all specimen didn't show obvious deformation. When the interstory drift increased to 0.03rad, the gap between the upper and the lower flanges of specimens CS-1 and CS-2 was 4mm and 5mm, respectively. the weld zones of the column foot for specimens CS-2 and WD-1 showed slight crack. As the interstory drift increased to 0.04rad, the gap between both flanges of specimens CS-1 and CS-2 was 5mm and 9mm, respectively. Meanwhile, the high-strength bolts of specimen CS-2 were sheared off. The cracks of the column foot for specimen WD-1 was furtherly developed. As the interstory drift reached 0.05rad, the gap between both flanges of specimen CS-1 grew to 7mm and the column foot of specimen WD-1 was bent. As interstory drift increased to 0.06rad, specimen



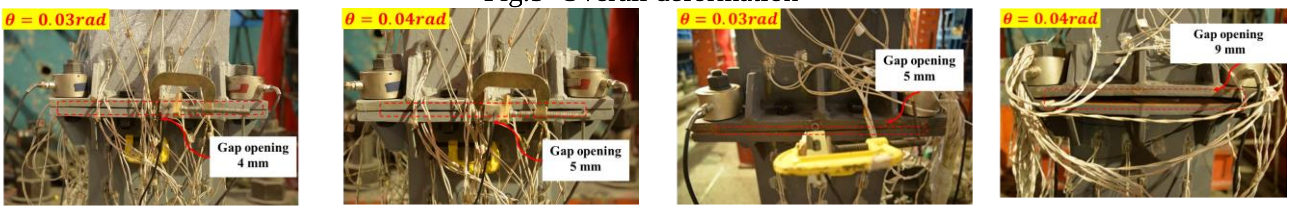
CS-1 damaged because the high-strength bolts were broken and specimen WD-1 was damaged due to the serious tearing of the column foot. The overall deformation of specimens and the gap opening between both flanges were shown in Fig. 5 and Fig. 6, respectively.

It can be concluded that the gap opening between both flanges for specimen CS-1 was obviously less than that of specimen CS-2 at each loading level, showing that the core tube can provide a certain bending bearing capacity to the connection and thus has reduced the tensile force of high-strength bolts.




(a) CS-1 (b) CS-2 (c)WD-1

Fig.5 Overall deformation



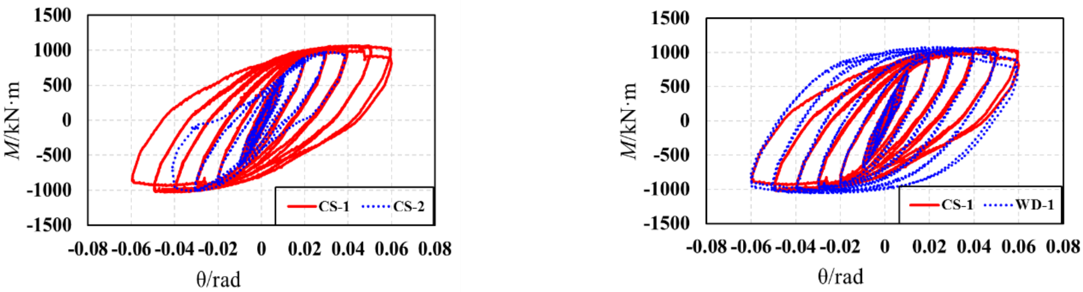

(b) CS-1 (a) CS-2

Fig.6 Deformation at the flange of the connection

### 2.3.2 Hysteretic curves

Fig. 7(a) shows the comparison of the relationship curves of the bending moment vesus the interstory drift (M-θ) for specimens CS-2 and CS-1. It can be concluded that the hysteretic curves of specimen CS-2 showed obvious pinching characteristics due to the fast development of gap openings between the upper and lower flanges as the interstory drift increased to 0.04rad and the lateral stiffness of the connection was relatively weak. However, Specimen CS-1 had presented better hysteresis performance with relatively fuller hysteresis curves. Therefore, the setting of the core tube can enhance the stiffness and improve the bending and shearing capacity of the connection.

Fig. 7(b) described the comparative conditions of the bending moment vesus the interstory drift (*M-θ*) for specimens CS-1 and WD-1. It can be seen that the hysteretic curves of both specimens are basically close to each other, which indicates that the fully-bolted column connection with core tube can achieve similar rigid connection to that of the welded connection.




（a）Comparison of specimens CS-1 and CS-2 （b）Comparison of specimens CS-1 and WD-1



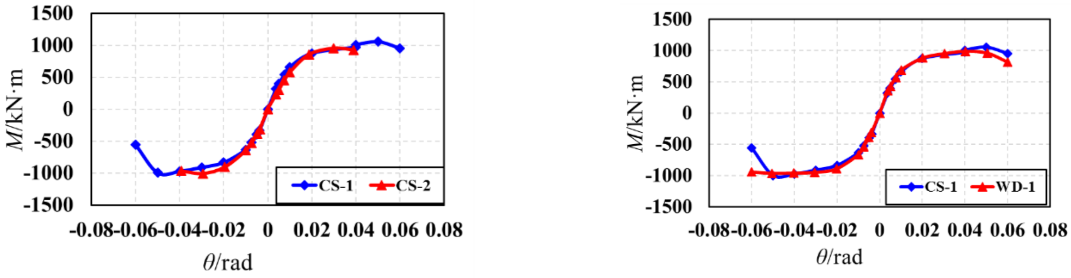
 

Fig. 7 Comparison of hysteretic curves

### 2.3.3 Skeleton curve

The skeleton curve is an envelope curve obtained by linking extreme bending moment points in both directions of the hysteresis curves. The comparative relations of the skeleton curves for each specimen were shown in Fig. 8. From the comparison of CS-2 and CS-1, it can be seen that the moment incrased 14% as the interstory drift was 0.01rad, and meanwhile the flexural capacity increased 2%, 3% and 8% under the loading leves of 0.02rad, 0.03rad and 0.04rad, respectively. Thus, it is indicated that the setting of core tube can improve the flexural capacity of the column connection.

Fig. 8(b) is the comparison of skeleton curves between specimens CS-1 and WD-1. It can be seen from the figure that the skeleton curves of the fully-bolted connection with core tube is close to that of the welded connection in the whole loading process. The average difference of peak bending moment between CS-1 and WD-1 under various loading levels was 2% as $\theta$ does not exceed 0.05rad. It was indicated that the fully-bolted connection with core tube has similar flexural capacity to that of the welded connection, which has effectively ensured the reliable connection between the upper and the lower columns.




（a）comparison of specimens CS-1 and CS-2 （b）comparison of specimens CS-1 and WD-1

Fig. 8 Comparison of skeleton curves

### 2.3.4 Ductility coefficient

The displacement ductility coefficient of a structure is defined to be the ratio of the ultimate displacement to the yield displacement of the specimen [12]. It can be obtained by universal bending moment method. The yield load $P_y$ , yield displacement $\Delta_y$ of all specimens were listed in Table 2 together with the peak load $P_m$, ultimate load $P_u$, ultimate displacement $\Delta_u$ and displacement ductility coefficient $\mu$. It can be known from Table 2 that the ductility coefficients of the specimen CS-1and WD-1 were both more than 3.0 which had good ductility. From the comparison between specimens CS-1 and CS-2, it can be found that the core tube can improve the ductility of the connection.

Table 2 Primary performance indicators of specimens during static tests

| Specimen number | Loading direction | $P_y$/kN | $P_m$/kN | $P_u$/kN | $\Delta_y$/mm | $\Delta_u$ /mm | $\mu$ |
|---|---|---|---|---|---|---|---|
| CS-1 | + | 423.71 | 530.23 | 479.39 | 36.57 | 118.30 | 3.23 |
| | - | 387.60 | 498.62 | 278.38 | 33.44 | 113.19 | 3.38 |
| CS-2 | + | 217.83 | 479.25 | 434.56 | 38.97 | 77.65 | 1.99 |
| | - | -209.99 | 503.61 | 418.96 | 34.04 | 78.29 | 2.30 |
| WD-1 | + | 196.40 | 496.09 | 391.82 | 29.53 | 100.89 | 3.42 |
| | - | -200.88 | 481.55 | 400.75 | 32.26 | 102.24 | 3.17 |

### 2.3.5 Energy dissipation capacities

Based on the abovementioned hysteretic curves, the energy dissipation of the specimen under each loading displacement can quantitatively calculated. The consumed energy versus the loading displacement was shown in Fig. 9.

It can be seen from Fig. 9 that the cumulative energy consumption of the specimens continued to grow as the displacement increased. As the specimens entered into elastoplastic state, the energy consumption of the specimens grew faster. Comparing with specimen CS-2, specimen CS-1 had better energy dissipation capacities. The final energy consumption of specimen CS-1 was about 2.7 times that of specimen CS-2.

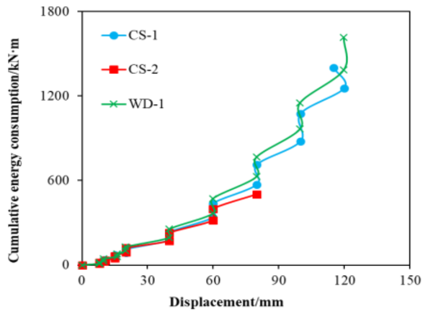
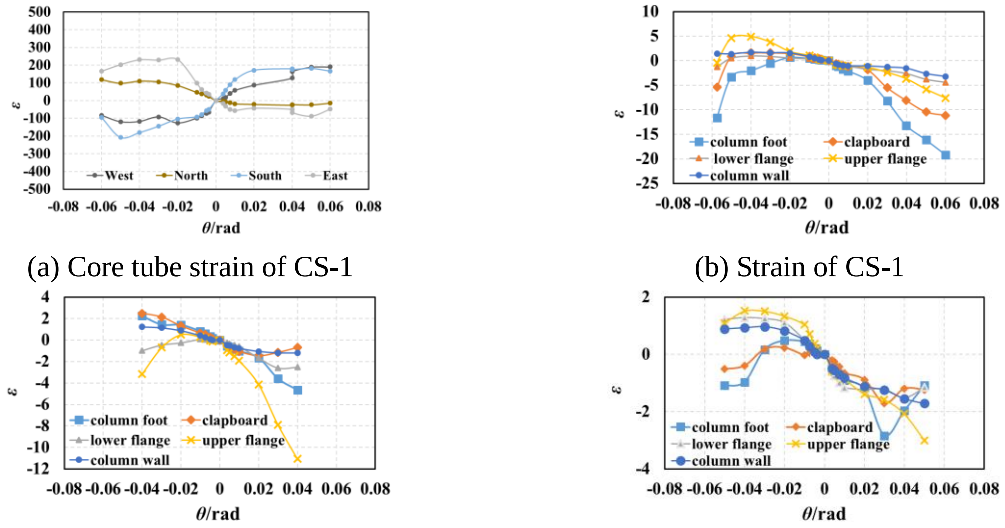



Fig.9 Energy consumption versus the loading displacement

### 2.3.6 Strain variation

Typical positions were selected for strain analysis, including core tube, column foot, clapboard, lower flange plate, upper flange plate, and upper column wall.The detailed strain variations were shown in Fig. 10. It can be known that the strain of the core tube had continued to increase in the whole loading process, which indicated that the core tube contacted the column wall and provided bending and shearing capacity. Under different loading levels, the maximum strain of the specimen CS-2 was at the upper flange, and then followed by that of the column foot, which indicated that the stiffness of the connection was less than that of the column body. The maximum strain of specimen CS-1 was at the column foot, which realized a rigid connection. The strain variation of the fully-bolted connection with core tube was nearly same as that of the welded connection. The strain of specimens CS-1 and WD-1 both increased from top to bottom gradually and the maximum value occurred at column foot. It was shown that the fully-bolted connection with core tube achived a rigid connection between the upper and lower columns, which had similar mechanical properties of welded connection.




(a) Core tube strain of CS-1

(b) Strain of CS-1

(c) Strain of CS-2

(d) Strain of WD-1

Fig.10 Strain changes

### 2.3.7 Bolt preload force

The preload force variation of high-strength bolts of specimens CS-1 and CS-2 during the loading process were shown in Fig. 11. Under different loading levels, the preload forces of high-strength bolts for specimen CS-2 was higher than that of specimen CS-1. It can be obtained that the preload forces of high-strength bolts were reduced by 46.4% as the interstory drift was 0.01rad, and were reduced by 32.6%, 32.4% and 24.7% as $\theta$ increased to 0.02rad, 0.03rad and 0.04rad, respectively. It can be seen that the setting of core tube can reduce the preload force of the high-strength bolts on the flange plates.



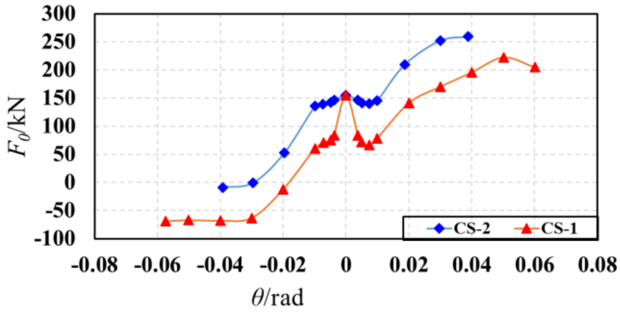



Fig.11 Preload force variation of high strength bolts for specimens CS-1 and CS-2

## 3 Dynamic shaking table tests on steel frames adopting fully-bolted connections with and without core tube

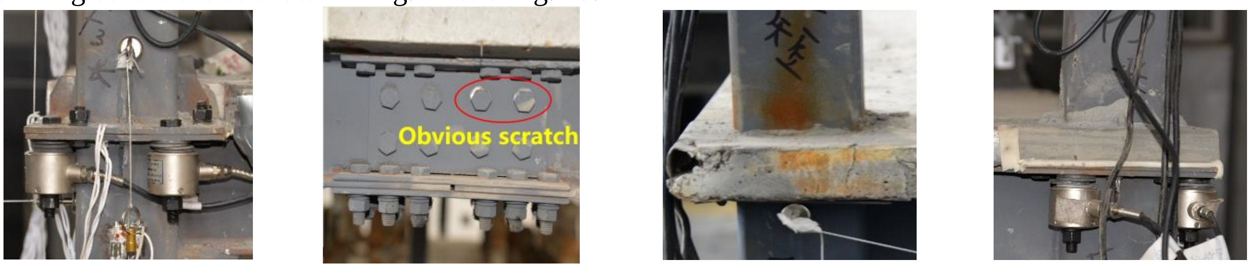
### 3.1 Details of shaking table tests

Based on model similarity theory, dynamic shaking table tests on two 1/4 scale, 5-storey spatial steel frames adopting fully-bolted connections with and without core tube have been conducted. Except for the set of core tubes, the other parameters of the two models were same.

Based on *Code for seismic design of buildings* [14], two natural seismic waves (Tangshan wave, EL-Centro wave) were selected and one artificial wave. During the tests, the loading sequence of Tangshan wave, EL-Centro wave and artificial wave was adopted. Testing phenomena of both steel frames were obsearved under various earthquake intensities with peak ground acceleration (PGA) varied from 0.07g to 1.0g.

### 3.2 Phenomenon and analysis of shaking table tests

Under the action of 8-degree rare earthquake (PGA=0.4g), slight scratches were found at the bolt splicing area of frame beams and slight cracks were obseaved on the floor slab of the frame without core tubes. On the whole, the two groups of frames had no obvious damage, and the typical positions kept good working conditions as shown in Fig. 12 and Fig. 13.




(a) Column joint zone (b) Beam conncting zone (c) Column joint grouting zone (d) Column joint concrete

Fig.12 Typial positions state of frame with core tubes under 8-degree rare earthqaukes (PGA=0.4g)

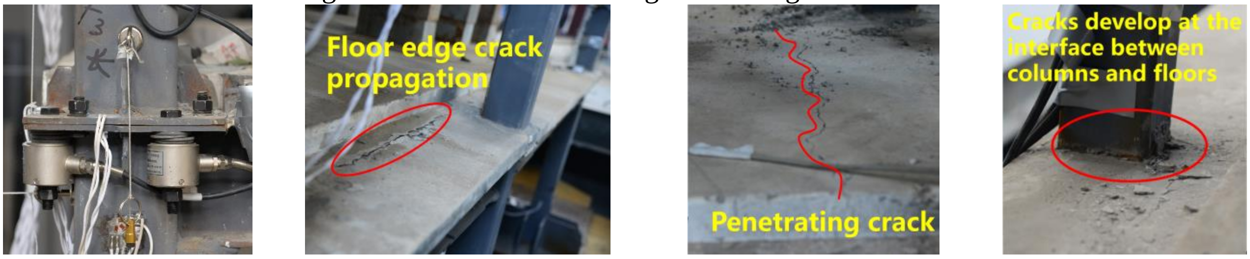

(a) Column joint zone (b) Beam connecting zone (c) Column joint grouting zone (d) Slab surface

Fig.13 Typical positions state of frame without core tubes under 8-degree rare earthquakes (PGA=0.4g)



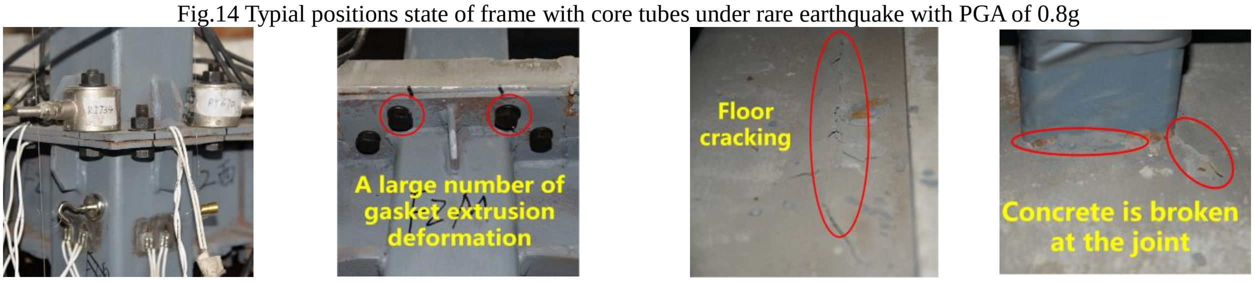
 

Under rare earthquakes (PGA=0.8g), the concrete floor slabs of two frames were generally broken, penetrating cracks appeared, and the concrete in the column joint area was severely broken. However, the connection with core tube had no obvious damage and maintained a stable and good working condition. The frame could continue to withstand higher magnitudes; the frame without core tubes could not be loaded due to the limit of vertical loading. Details were shown in Fig. 14 and Fig. 15.




(a) Column joint zone (b) Floor slab edge (c) Column joint grouting zone (d) Column joint concrete

Fig.14 Typial positions state of frame with core tubes under rare earthquake with PGA of 0.8g




(a) Column joint zone (b)Column joint bolt shim extrusion (c) Slab cracks (d) Column joint concrete

Fig.15 Typial positions state of frame without core tubes under rare earthquakes with PGA of 0.8g

## 4 Conclusion

1) The core tube can provide certain bending and shearing capacity for the column connection. When the interstory drift varied from 0.01 rad to 0.04 rad, it could increase the bearing capacity of the column connection by 2% to 14% and reduce bolt tension by 24.7% to 46.4%. The cumulative energy consumption of connection with core tube was 2.7 times that of connection without core tube. The setting of the core tube effectively delayed the destruction of the column connection and strengthened the energy dissipation and ductility of the fully-bolted column connection.

2) The fully-bolted connection with core tube had similar mechanical properties with the welded connection. When the interstory drift does not exceed 0.05rad, the average difference of peak bending moment between the two types of connections under various loading levels was 2%. The ductility coefficients were both more than 3.0 and close to each other. It can be designed and calculated according to rigid connections on the basis of achieving efficient assembly of columns.

3) Shaking table tests indicated that the steel frame with core tubes had less damage. The core tube can effectively resist the bending and shearing force at the connection zone. The connection in the frame without core tubes lost rigid connection performance under extremely rare earthquakes and cannot provide stable rigidity to the entire frame.The fully-bolted connection with core tube had no obvious damage and showed typical rigid connection.



## 5 Reference

[1] SUMNER E A , MURRAY T M . Behavior of extended end-plate moment connections subject to cyclic loading[J]. Journal of Structural Engineering, 2002, 128(4):501-508.

[2] Hwang H J, Eom T S, Park H G, et al. Cyclic Loading Test for Beam-Column Connections of Concrete-Filled U-Shaped Steel Beams and Concrete-Encased Steel Angle Columns[J]. Journal of Structural Engineering, 2015, 141(11):04015020.

[3] LIU X.C, XU A.X, ZHANG A.L, NI Z, WANG H.X, WU L. Static and seismic experiment for welded joints in modularized prefabricated steel structure[J]. Journal of Constructional Steel Research,2015,112.

[4] ZHANG Ailin, ZHAO Yue, NI Zhen, et al. Experimental research of flexural behavior of the truss girder in modularized prefabricated steel structures[J]. Industrial Construction, 2015(10):18-23.

[5] LUE Xuechun, XUAxin, ZHANG Ailin, et al. The static performance analysis and experimental research of the joint in modular prefabricated high-rise steel structure[J]. Industrial Construction, 2014,44(8):27-34.

[6] LIU Xuechun, XU Axin, NI Zhen, et al. Analysis of the limit bearing capacity and seismic performance of typical joint of truss-beam and column in fabricated high-rise steel structure[J]. Industrial Construction, 2014,44(8):23-26.

[7] LIU Xuechun, PU Shuanghui, XU Axin, et al. Experimental study on static and seismic performance of bolted joint in modularized multi-layer and high-rise prefabricated steel structures[J]. Journal of Structural Engineering, 2015, 36(12):43-51.

[8] LI D , UY B , PATEL V , et al. Behaviour and design of demountable CFST column-column connections subjected to compression[J]. Journal of Constructional Steel Research, 2018, 141:262-274.

[9] Zhang Yanxia, Huang Weizhen, Zheng Mingzhao, Wang Yan, Ning Guang. Quasi-Static Experimental Research on Box-Section All-Bolted[J]. Industrial Construction, 2018,48(5) : 37-44

[10]Zhang Yanxia, Zheng Mingzhao, Huang Weizhen, Jiang Kun, Ning Guang. Determination of structure ductility coefficient[J]. Progress in Steel Building Structures, 1980,10(5) : 64-65

[11]Zhang Ailin, Shao Dinan, Zhang Yanxia, Zheng Mingzhao, Ning Guang, Parametric Analysis of Box-section All-boited[J], Industrial Construction, 2018, 48(5) :45-53.

[12]Spcification for seismic test of buildings: JGJ/T 101-2015 [S].Beijing:China Architecture & Building Press,2015.

[13]Specification for Structural Steel Buildings:ANSI/AISC 360-10[S].USA: America Institute of Steel Construction. AISC,2010

[14]Code for seismic design of buildings GB50011-2010[S].Beijing: China Architecture & Building Press,2010.

