Experimental Studies on Monotonic Behavior of Concrete-Filled Steel Square Tubular Column-Steel Beam Connection

Fan Hong[1] and Lihua Xu[2]

[1] *PHD, China Nuclear Power Company,LTD, Shenzhen. China*
[2] *Professor, School of Civil Engineering, Wuhan University, Wuhan. China*
Email: fanhong360@163.com, xulihua-d@126.com

ABSTRACT :

Based on static experimental results of four specimens of concrete-filled steel square tubular column and steel beam connections, the paper investigates the mechanism and failure mode of this kind of connection, and analyzes some factors, including installation of plates to both sides of flanges, the width-to-thickness ratio of columns and the length of the diaphragms, which have influences on the bearing capacity and load transfer ability of connections. The experimental results show that this type of connection has the feature of clear loaded and stress transfer pathway; The changes of the width-to-thickness ratio have little influence on the bearing capacity of a connection, but have a little influences to the stains of the gusset plant. The stress distributions can be ameliorated and plastic hinges are located outside of the brittle weld zones and appear on the beam through welding additional plates to both sides of flanges, so that the ductility of the connection can be enhanced.

KEYWORDS: Square Concrete-filled Steel Tubular Column; Connection; Mechanism; Static behavior

1. INTRODUCTION

Concrete-filled steel tube columns are gaining popularity all over the world in the building construction industry. For CFST columns make good use of two kinds of materials under load, this structure system displays many advantages, such as simply connection section, high bending strength, good stable performance, convenient for taking fire precautions. And in the past few years a large number of studies had been carried out on the performance of CFST columns through test and finite element analyses, such as Zhong[1], Han[2], A.Elremaily[3], K.A.S Susantha[4].et al.

In general, in this type of construction, steel beams are framed to the CFST columns at each floor level. Recently, because of the requirement for practical application, many research works were carried out in the related connections field. A.Azizinamini et al[5] tested six large scale connection specimens and seven, two-thirds scale connection specimens which composed of circular CFST columns and steel beams to comprehend the force transfer mechanisms between steel beams and CFST columns. Isao et al[6] studied nine subassemblies under simulated seismic loading. All specimens consisted of two types, seven were interior type subassemblies and two were exterior subassemblies, both square and circular column cross sections were examined. Lv[7] investigated the interior diaphragm connections and proposed the calculation method to evaluate the shear strength of the panel zone which was accepted by the CECS. Nie et al [8] also studied the interior diaphragm connections with three inner diaphragms and characterized the method for calculating the bending capacity. Chen, Yuan, et at[9,10] conducted analytical and experimental studies on external diaphragm connections.

Compared with the past studies, the research work of the inner and through-type diaphragm connection studied in this paper is very few. So far, only a few experiment studies were carried out, such as Cheng[11], Fujimoto[12]. This type connection is recommended by Technical Specification for Structures with Concrete-filled Rectangular Steel Tube Members (CECS 159:2004). Compared with the connection of the internal diaphragm, this type can solve the welt difficulty, which lies in the internal diaphragm connections caused by the small size of the steel tubular column, and avoid to product more residual stress caused by welding the flange of the beam and the internal diaphragm at the same position of one side of column, which can improve the ductility and the resistance behaviors of the connection. In order to promote this type connection used in actual project, the monotonic tests of four connections were carried out in this paper. Some factors were discussed including the additional plates to both sides of the flanges, the width-to-thickness ratio of the column and the length of the through diaphragm.

2. EXPERIMENT GROGRAM

2.1 Subassemblies Design and Manufacture

All specimens used in this study were 1/2 scale, planar cruciform connections, the connection type of specimen SZ61was recommended by the Specification. The beams of specimens were composed Ⅰ -beams, and the columns were the cold bending hollow square shape steel pipes and the concrete grade was C40. All steel used were of grade 235MPa minimum. The diaphragm located the position of the beam flange and went through the column. The column and beam flanges were connected to the diaphragm via full-penetration double-V-groove welds. The wet of beam was attached to the web cleat plate via six bolts, and the gusset plant was welded to the column via continuous fillet welds. The axial force ratio of every specimen was constant. The parameters investigated in the study include the additional plates to both sides of the flanges, the width-to-thickness ratio of the column and the length of the through diaphragm. The connection type that was the focus of this experimental program shown in Fig.1 and Table 1 summarizes the test matrix containing these specimens. The material properties were listed in Table 2.

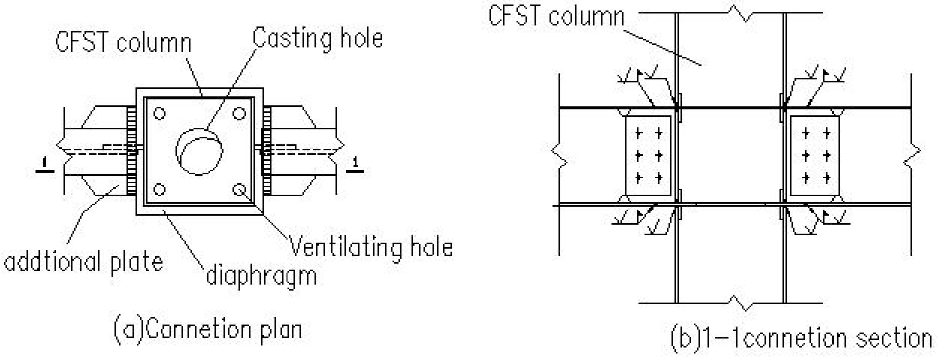


Fig. 1 The connection configuration

Table 1 Specimen description

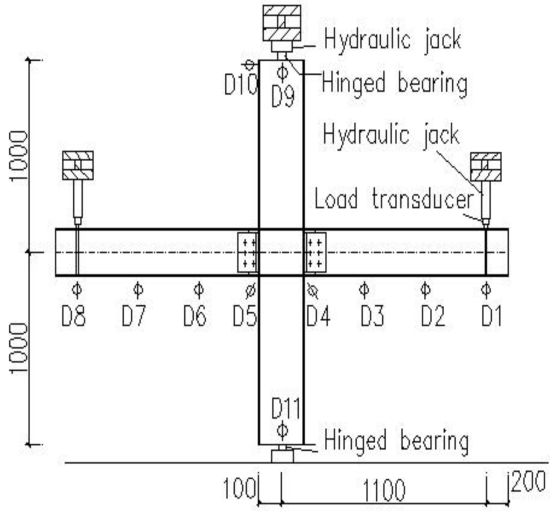
Specimen	Column section	Beam section	Additional plate size	The length of diaphragm	Concrete grade
SZ61	200×200×6	260×140×5×6	—	60	C40
SZ62	200×200×6	260×140×5×6	260×50×6	60	C40
SZ81	200×200×8	260×140×5×6	260×50×6	60	C40
SZ82	200×200×8	260×140×5×6	260×50×6	40	C40

Table 2 Material properties

Element	Yield strength f_y (N/mm^2)	Ultimate strength f_u (N/mm^2)	E_s (N/mm^2)
Beam flange	331	462	1.965×10^5
Beam web	350	468	1.938×10^5
Column（6mm）	322	428	1.953×10^5
Column（8mm）	310	413	1.942×10^5
Concrete compressive strength value		38.5 MPa	

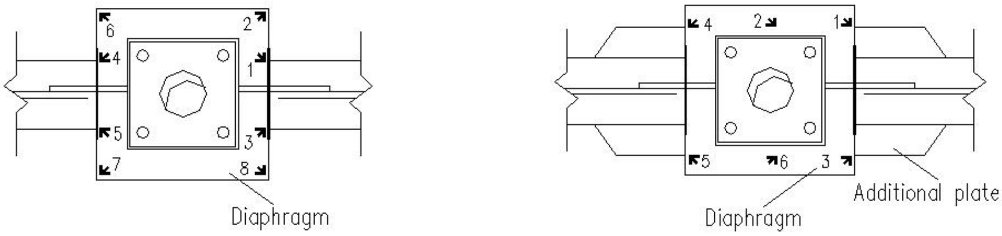
2.2 Instruments layout

Each specimen was fitted with a variety of devices to record the beam displacements and the strains of beam, diaphragm and gusset plant. Eight displacement transducers (D1～D8) were placed symmetry along the beam length to record the beam displacement. And three displacement transducers (D9～D11) were placed at the column end to monitor whether the connection was transfer under loading. In order to comprehend the transfer mechanism and observe the change of connection internal force, strain gauges were also placed on the beam flange and web, diaphragm and gusset plate. The load, displacement and strains were continuously collected by the instrument DH3815. The instruments measured points and the specific layout were shown in Figure 2 and 3.



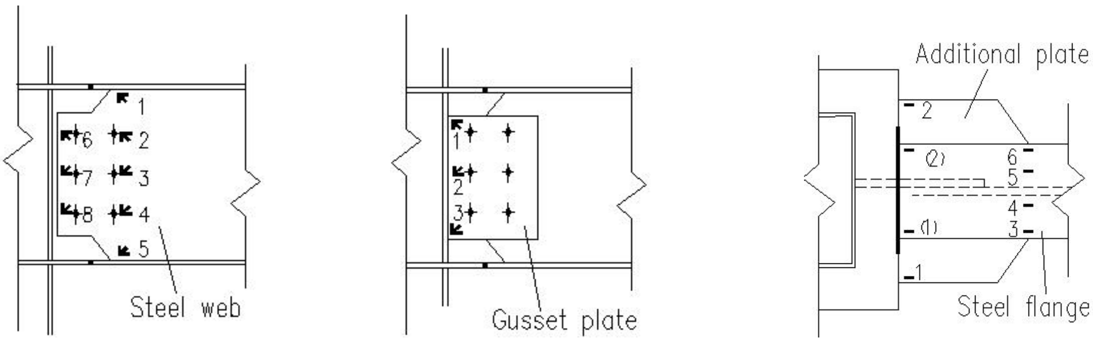
(D2～D7 were electron centesimal meter and the others were displacement transducers)

Fig.2 The layout of instrument



(a)Layout of the strain gauges of specimen SZ61

(b) Layout of the strain gauges of the rest specimens



(c) Layout of the strain gauges of steel web

(d) Layout of the strain gauges of gusset plate

(e) Layout of the strain gauges of steel flange

Fig.3 Layout of the strain gauges

2.3 Test procedures and loading history

Each connection specimen was loaded as depicted in Fig.2. When the specimen was seated, it must be noticed that the loading spot and orientation should be in the specimen axis, and it must be assured that the specimen should be loaded in the plane. Considered the ability of experimental equipments and the aim of this experiment, The axial force ratio of every specimen was 0.2 through calculating. We should preload before the formal experiment, so that each part of the specimen was contact well and the force-displacement curve went to be stable when the specimen was in the normal state, and this procedure can inspect whether the set-up were all reliable and the instruments were in the normal state. It was departed into three steps when preloading, then unloading step by step. It was firstly loaded on the concrete-filled steel tubular column step by step using a hydraulic jack. Then the column load was kept constant, and a purely monotonic load was applied at the beam tip until connection failure. Every step load should last fifteen minutes during loading in order to make the deformation develop sufficiently and reach stably.

3 EXPERIMENT RESULT AND ANALYSIS

3.1 Failure mode

For specimen SZ61, the column deformation was very small when it was loaded. The deformation was not visible and can be seen through the change of displacement sensor. But the beam deformation was obvious loading on the beam. When the load reached to 106kN, the specimen had a visible yield of the compressive flange connecting with the diaphragm. The failure mode was showed in Fig.4. The displacement was increasing quickly, and the beam reached the ultimate bearing capacity and the test was terminated.
For specimens SZ62, SZ81 and SZ82, the additional plates were added on the beam flanges. These specimens had similar phenomena with specimen SZ61. Deformation of column was very small and the beam deformation was very obvious. These specimens had the capacity to form a full plastic hinge in the variety section of beam flange. The failure mode was referred to Fig.5.
No signs of distress were evident at the diaphragm during loading for all specimens, and it was not found that the weld had cracked or the face of column was raised or sunken after testing.

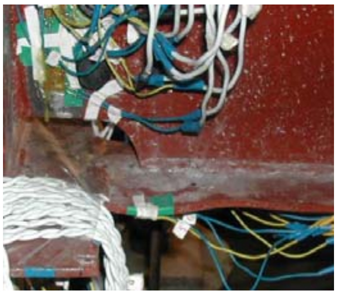
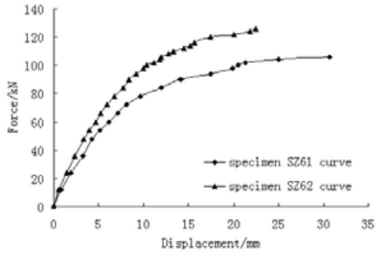


Fig 4 Flexure failure on the compress flange of beam end

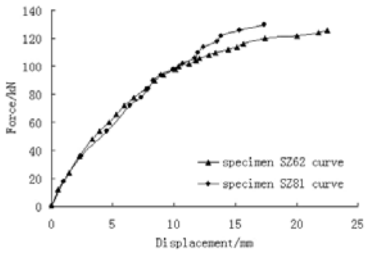
Fig 5 Flexure failure on the compress flange of changed section

3.2 Result analysis

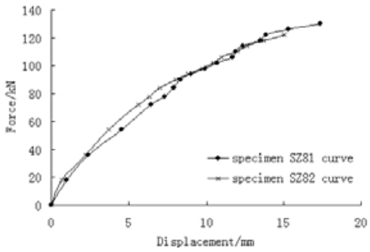
3.2.1 The load-displacement curve



(a) curves of specimens SZ61 and SZ62



(b)curves of specimens SZ62 and SZ81



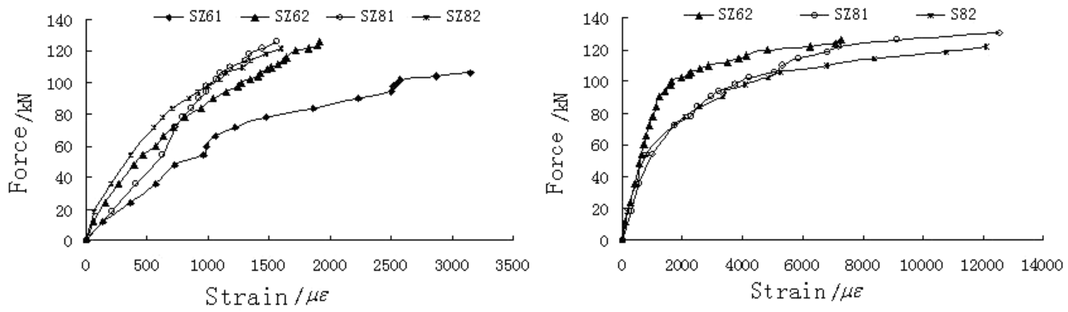
(c)curves of specimens SZ81 and SZ82

Fig 6 Load versus displacement curves

It can be concluded from analyzing the Fig6: (1) The ultimate bearing capacity was improved to 25% and the beam end displacement was reduced to 40% after adding the additional plates. (2)The change of the length of diaphragm and the width-to-thickness ratio had little influence to the increase of ultimate bearing capacity.

3.2.2 Beam strain analysis

Through analyzing the average strains of the measure points 1#～2#and 3#～6# of steel beams, we can obtain the load-average strain cures(refer to Fig.7). it was indicated that: (1)When the specimens reached the ultimate bearing capacity, the average strains of specimen SZ61 points 1#～2#reached 3500 , while the average strains of others specimens reached 2000 , but the average strains of points 3#～6# of the specimens SZ62, SZ81andSZ82 already reached 8000 . The phenomena were coincident with the failure modes of these specimens. The fracture of the specimen SZ61 was in the section where points1#～2# were located, while the others had the damage at the variety section where points 3#～6# were located. It was shown that the connection stiffness was strengthened and the plastic hinge was out of the panel zone after adding the additional plates, so that the connection can satisfied the design requirement “strong joints and weak components”. (2) The load-strain curves of the specimens SZ62, SZ81and SZ82 were all familiar, which showed the length of through diaphragm and the width-to-thickness ratio had little influence to the beam capability.



(a) load versus average strains curves of points1$^{\#}$and 2$^{\#}$ (b) load versus average strains curves of points3$^{\#}$～6$^{\#}$

Fig 7 Load versus average strains curves

3.2.3 Diaphragm strain analysis

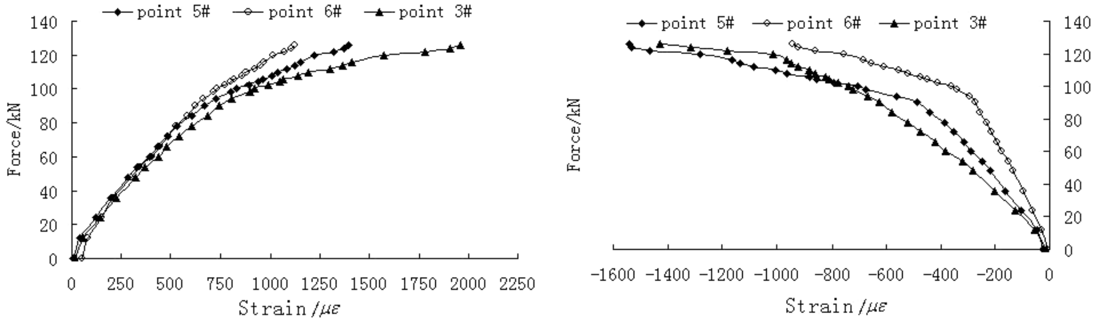
For this type connection, the diaphragm was mainly subjected to the bending moment, so the diaphragm was the main objective when studying the connection mechanism. Since load transferring path of the diaphragm was very complex, we took the strain rosettes to monitor the change of diaphragm. We can conclude following regulations through analyzing these primary strains:
(1)The primary tension strains of the upper diaphragm adjacent to the beam were list in the table 3 under every level load.

Table 3 the principle tension strains of diaphragm under every lever load

load	SZ61		SZ62		SZ81		SZ82	
	1$^{\#}$	3$^{\#}$	1$^{\#}$	3$^{\#}$	1$^{\#}$	3$^{\#}$	1$^{\#}$	3$^{\#}$
60kN	1764	1761	622	639	1120	1186	1050	1095
90kN	5523	5692	1055	1107	1426	1457	1401	1373
106kN	—	—	1625	1957	2213	2206	2131	2066
Ultimate load	—	—	2861	2937	3267	3378	3173	2993

From the table 3, we can see that there was a stress concentration phenomenon in the conjunction of the diaphragm and the beam for the specimen SZ61, while the concentration phenomenon can be decreased after adding the additional plates; the change of width-to-thickness ratio have some influence to the diaphragm’s capacity, and the primary tension strains was less little when the ratio was more bigger under the same load.

This maybe was because the concrete-filled steel tubular column was in the compressive state during loading on the beam, then the some tension to the diaphragm may be induced by the column, and the tension would be less if the width-to-thickness ratio were bigger, thus the total tension stress of the diaphragm would be less; The primary tension strains of the specimens SZ81 and SZ82 were very close, which was shown the length of the through diaphragm had little influence to the diaphragm's capacity.
(2)In order to study the mechanism of the connection diaphragm we analyzed the tension strains of points 3#，5#，6# on the upper and under diaphragms. The force-primary strain curves of the points on the same location of tension diaphragm and compression diaphragm of specimen SZ62 were shown in the Fig.8. The curves of points 3#，5#，6# on the tension diaphragm showed that three curves were very close, this was because the width of the diaphragm and the width of the flange were familiar after adding the additional plates on the beam end, thus the tension induced by the beam moment was distributed symmetrically on the diaphragm, while the force-primary strain curves of the points on the compressive diaphragm were different , this was because the core concrete was subjected to the partial compressed load so that the primary strains of middle point were smaller than those of the side points.



(a)the curves of the points on the tension diaphragm (b)the curves of the points on the compression diaphragm

Fig 8 load versus strain curves

(3) For the specimen SZ61, the primary strains of points 2#and 8# on the diaphragm were 295 and 231 respectively when it reached the ultimate load, while the primary strains of points 1# and 3# located in the same section with the points 2#and 8# were very large. It was maybe because there was a stress concentrate phenomenon owing the different width between the flange and the diaphragm, and the around material of diaphragm had not a good use, so we should change the diaphragm structure through adding the additional plates or other measures in the actual project. For the diaphragm structure, the paper advised to adopt the type recommended by Reference 10.
In summary, we can conclude that the stress concentration phenomenon can be released and make the stress distribute symmetrically after adding the additional plates at the beam ends; the width-to-thickness ratio had some influence to the diaphragm's stress, but the length of through diaphragm had little influence to the diaphragm's capability.

3.2.4The beam web strain

In order to study the shear distribute regulation of beam web, table 4 had listed shear strains of eight points on the steel beam web under every lever load of specimen SZ61.

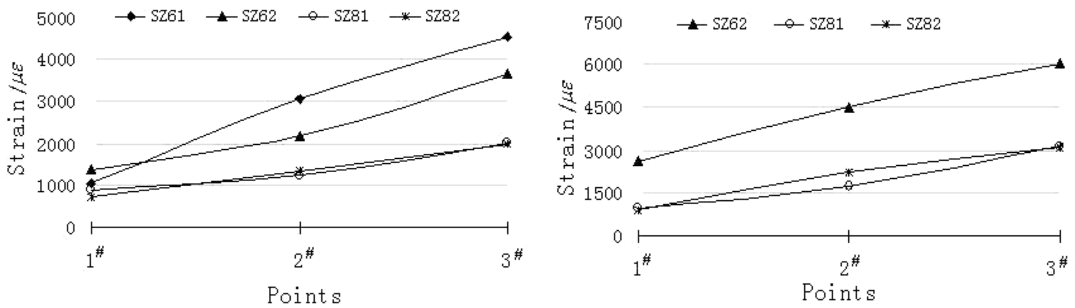
Table 4 shear strains of the steel beam web

load（kN）	1#	2#	3#	4#	5#	6#	7#	8#
48	463	192	165	143	362	41.6	51.6	22.1
60	604	262	213	180	528	27.5	52.5	9.2
90	1318	355	292	202	1062	49	59.9	27.9
106	1772	462	261	159	1112	72	68.4	24.2

From the table 4, we can conclude that: the shear strains of points 2#，3#，4# were smaller than those of points 1#，5#. The points 2#，3#，4# were around the bolts, so they were subject to the shear together, while the points1#，5# were out of the bolts zone, and they were subject to the shear alone. With the load increasing the difference strains value between the points 2#～4# and 1#，5# became more and more great, which showed the bolts were subject to the shear mainly. The shear strains of points 6#～8# located at the edge of web were smaller that those of points 1#～5# which showed the shear was almost transferred to the gusset plate. The mechanism of points in the bolts zone was complex and the strains changed confusedly. But we still judged the shear transferred path clearly through the strain change regulations of the eight points.

3.2.5 Gusset plate strain

The gusset shear strains of the specimens were shown in the Fig.9 under the 106kN (the ultimate load of specimen SZ61) and 122kN (the ultimate load of specimen SZ82). Compared the data we can see the width-to-thickness ratio and the additional plates had some influence to the gusset plate strains, while the length of through diaphragm had little influence to the gusset plate strains. At point 3#, the difference of shear strains value between specimen SZ61and specimen SZ62 was 20%， and the difference between specimen SZ62 and SZ81 was 45%， which showed the influence of the width-to-thickness ratio was greater than that of the additional plates, and the tendency was that the ratio was greater the shear strain was smaller. It was because the shear of gusset was transferred to the column face through welt, and the thickness of column was bigger, the shear transferred to the column was more, while the shear strains of gusset were smaller.



(a)the shear strains of each point under load 106kN (b)the shear strains of each point under load 122kN

Fig 9 shear strain comparison of the gusset plant

4 THE BENDING MECHANISM OF CONNECTION

For this inner and through-type diaphragm connection, the moment transferring depended on the beam flange before the moment was transformed into the panel zone and it was depend on the diaphragm after the moment was transformed into the panel zone. The bending capacity of the connection was decided by the bending capability of beam, the connection welt between the flange and diaphragm, and the diaphragm.

For the tall buildings with the concrete-filled steel tubular column and steel beam, it should satisfy the design requirements “strong column weak beam and stronger joints” and some relevant requirements. If the frame connection satisfies the relevant requirements, the tension capability of diaphragm in the panel zone is greater than that of the beam flange. No signs of distress were evident at the welt welded with diaphragm and no signs of yield were found after experiment, so the bending capacity of panel zone is decided by the bending capacity of steel beam. If the beam capacity is sure the bending capacity of panel zone can be satisfied. According to the computed formula recommended by the Code of Steel Structure Design (GB50017-2003), and using the material properties obtained in this experiment, the bending capacity of the specimens in the beam or variety section were calculated. The computed values and measured values of the ultimate load for the specimens were listed in the table 5.

Table 5 compared the ultimate loads of experiment with those of analysis

specimen	Computed value P_b/kN	Measured value P_u/ kN	P_u/P_b
SZ61	95.13	106	1.11
SZ62	131.50	126	0.96
SZ81	131.50	130	0.99
SZ82	127.74	122	0.96

We can see that the differences between the computed value and measured value were very small, especially the latter three specimens, the differences were in 5%, which show the connection bending capacity was decided by that of the steel beam, and this was agree with the specimens fracture.

5 CONCLUSIONS

(1) The transferring path of this type connection-inner and through diaphragm is very clear and each part is definitely subjected to dint, easy to calculation and design.
(2) The specimen recommended by the specification has appeared a serious stress concentration phenomenon, and this phenomenon can be eased through adding the additional plates. This method can make the plastic hinge away from the brittle weld zone and appear on the beam, so that the ductility of connections can be enhanced.
(3) The change of the width-to-thickness ratio of the concrete-filled steel tubular column has little influence to the bending capacity. And the ratio is greater the shear strains of gusset plates are greater.
(4) The length of through diaphragm has little influence to the connection behaviors.
(5) The connection bending capacity is decided by the beam bending capacity, and the calculation value and measured value of steel beams are agree with well.

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