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Yufen Zhang
Degang Guo

Structural Analysis of Concrete-Filled Double Steel Tubes



Science Press
Beijing



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ISSN 2366-259X ISSN 2366-2603 (electronic)
Springer Tracts in Civil Engineering
ISBN 978-981-15-8088-8 ISBN 978-981-15-8089-5 (eBook)
<https://doi.org/10.1007/978-981-15-8089-5>

Jointly published with Science Press

The print edition is not for sale in China (Mainland). Customers from China (Mainland) please order the print book from: Science Press.

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Preface

The survey of development for Concrete-Filled Steel Tubes (CFST) is briefly introduced. Since the late 1980s, two new application fields of CFST were opened up in China, which are highway, city arch bridges or tall or super tall buildings. Columns are required to increase not only high bearing capacity but also good ductility, so this monograph represents one new type of reinforced CFST called Concrete-Filled Double Steel Tubes (CFDST). Understanding is based on explaining the physical behavior of CFDST under load and then modeling this behavior to develop the theory. The authors have systematically studied the CFDST structure for more than 10 years, and this monograph is intended to present the work completed by the authors and their coworkers only. The structural theory is based on connecting and linking it to real buildings and structural components. This monograph presents the results of new loading tests and numerical simulations to investigate the mechanism of the CFDST column and the beam-to-column connections, and the effectiveness of changing different construction details.

Organizationally, the monograph consists of two parts: column performances in Chaps. 2 and 3, and joint constructions covered in Chaps. 4–9. A heavy emphasis is placed on the performance analysis of the beam-to-column connection in understanding the forces transferring mechanism. All problems begin with structural designs and are accompanied by experimental studies or numerical simulations. Chapter 1 introduces the development and application of CFST, and a general introduction to its connection forms is also included. CFDST is well within the range of reinforced CFSTs to meet the requirements of higher bearing capacity and better performance. Chapter 2 uses a method of Unified Theory of CFST and an elastoplastic limit equilibrium method to calculate the compressive strength of the composite columns, with an attempt to examine the optimal cross-section and parameter options. Chapter 3 uses the principles discussed in Chap. 2 to solve the problem of axial stiffness of the CFDST column. The process of structural design in Chap. 4 illustrates the applicable beam-to-column connections in CFDST structures without more accompanying calculations and analyses. Chapters 5–9 develop the basic design and mechanical performance for five types of beam-to-column connections including Ring beam joint, External diaphragm joint, Vertical stiffener joint, Anchored web

joint, and Blind bolted joint. To the end, the main research conclusions and the relevant future work are also discussed in Chap. 10.

Throughout the monograph, many analyses and design applications are presented, which involve mechanical elements and structural members often encountered in engineering practices. This monograph is written for practicing structural and civil engineers, students, and academic researchers, who want to be familiar with the latest technologies for CFST structure. This monograph should, therefore, be an invaluable resource for lecturers, graduate students in structural, architectural and civil engineering, explaining the important principles in the behavior of tubular steel structures. It is also addressed to designers of steel structures, who can find in it the special items related to the use of steel connectors, in particular joints, their failure modes and analytical models as supplements to more general design codes.

Readers are welcome to point out any errors noticed in the monograph so that corrections can be incorporated. There must be some limitations and errors in the contents. Any suggestions are welcome by the authors.

Tianjin, China
Beijing, China
March 2020

Yufen Zhang
Degang Guo

Acknowledgements

With respect to preparing the manuscript for this monograph, I gratefully acknowledge the pleasant and effective cooperation with Project Manager Degang Guo, who was responsible for the relevant experimental process. This monograph covered the research outcomes of the National Science Foundation of China (Grant No. 51478004&51008027). Meanwhile, the financial support from Hebei University of Technology, Norinco International Cooperation Ltd. and State Key Laboratory of Hydrosience and Engineering are also appreciated.

The work presented in this monograph is completed by other co-workers, including Prof. Yimin Song of North China University of Technology; Drs. Dongfang Zhang, Su Wang, Mr. Wanji Pei of Chang'an University. I would also like to thank all my graduate students, who have contributed extensively to the development, analysis, and simulation work presented in this monograph. They include Qiang Wang, Fu Wang, Xinxiu Wu, Ge. Zhu, Zonghao Jiang, Miao Wang, Yushuo Li, Hongxin Jia, Jiaqi Gao, Jinxin Hou, Aer Pan. During the writing up stage, Mr. Xuwang Zhao and Ms. Yan Zhang helped to finalize all the figures and integrate the text; Mr. Hongfan Bu, Mr. Evans Awua-Peasah, and Ms. Benedicta Be diako-Poku helped to proofread the entire manuscript. Their contributions are gratefully acknowledged.

A particular note of thanks goes to my graduate advisor, Prof. Junhai Zhao of Chang'an University, who has been a great help to me in this regard. Also I thank Profs. Guowei Ma, Xian Rong of Hebei University of Technology, Prof. Jianguo Nie of Tsinghua University (Member of the Chinese Academy of Engineering), who provided generous support to this research.

Finally, appreciation goes to my husband and my daughter who have been a source of encouragement.

March 2020

Yufen Zhang

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Symbols

A	Cross-sectional area
A_a	Shear area of the anchorage web
A_c	Cross-sectional area of concrete
A_{cf}	Cross-sectional area of CFRP tube
A_{ci}	Cross-sectional area of inner concrete
A_{co}	Cross-sectional area of outer concrete
A_m	Cross-sectional area of confining material
A_{ot}	Cross-sectional area of the outer steel tube
A_s	Cross-sectional area of steel tube
A_{sc}	Cross-sectional area of CFST column
A_{si}	Cross-sectional area of inner steel tube
A_{so}	Cross-sectional area of outer steel tube
A_{ssc}	Cross-sectional area of CFDST column
A_s	Cross-sectional area of steel tube web
A_v	The horizontal cross-sectional area of the continuous vertical stiffener
a	Weighted coefficient (reflects contributions of confining materials)
B	Side length of square steel tube
B_c	Coefficients related to the tensile strength
b	Weighted coefficient (reflects contributions of concrete)
b_{ep}	Width of T-plate
b_f	Overall width of the I-shaped beam
C	Material strength
D	Diameter
D_o	Diameter of equivalent circular steel tube
D_i	Diameter of inner steel tube
d	Compressive/Tensile damage coefficient
d_c	Compressive damage coefficient
d_t	Tensile damage coefficient
E	Elastic modulus of CFST column
$E_b I_b$	Elastic bending stiffness of the steel beam

E_c	Elastic modulus of concrete
$E_c I_c$	Elastic bending stiffness of the CFDST column
E_o	Elastic modulus of CFDST using simple superposition method
E_s	Elastic modulus of steel
E_{sc}	Elastic modulus of CFST column
E_{ssc}	Composite elastic modulus of the CFDST column
E_t	Half-cycle energy dissipation
E_{total}	Total dissipated energy
$\bar{E}_{si}$	Modified elastic modulus of inner steel tube
$\bar{E}_{so}$	Modified elastic modulus of outer steel tube
$\bar{E}_c$	Modified elastic modulus of concrete
e_f	Distance from bolt center to the web edge of T-plate
e_x	Distance from bolt center to flange edge of T-plate
f	Strength of material
f_c	Standard compressive strength of concrete
f_c'	Strength of concrete cylinders
f_{cf}	Strength of CFRP
f_{ck}	Standard value of axial compressive strength of concrete
f_{co}	Compressive strength of outer concrete
f_{ci}	Compressive strength of inner concrete
f_{cu}	Ultimate compressive strength of concrete
f_m	Yield strength of confining material
f_s	Steel strength
f_{si}	Strength of inner steel tube
f_{so}	Strength of outer steel tube
f_{ss}	Weighted average yield strength of confining material
f_{ssc}	Compressive strength of CFDST column
f_{sc}^p	Nominal composite proportional limit stress of CFST column
f_{ssc}^p	Proportional limit stress of CFDST column
f_{sy}	Yield stress of steel
f_{su}	Ultimate shear strength of steel tube
f_t^w	Tensile strength of the weld seam
f_{vu}	The ultimate strength of vertical stiffener
f_v^w	Shear strength of the weld seam
f_u	Ultimate strength
$f_{u,eb}$	Ultimate strength of T-plate
f_y	Yield stress
f_{co}	Compressive strength of outer concrete
$\bar{f}_{co}$	Actual ultimate strength of outer concrete
f_{ci}	Compressive strength of inner concrete
$\bar{f}_{ci}$	Actual ultimate strength of inner concrete
f_{so}	Strength of outer steel tube
$\bar{f}_{so}$	Actual ultimate strength of outer steel tube
f_{si}	Strength of inner steel tube
$\bar{f}_{si}$	Actual ultimate strength of inner steel tube

$g(\cdot)$	A function of the concrete volume
H_b	Beam height
H'	Distance between the lateral load acting point and the hinge center
H'_{bottom}	Distance between bottom hinge center and bottom surface of the steel beam
H'_{top}	Distance between lateral load acting point and top surface of the steel beam
h	Overall height of the steel beam
h_a	Rib height of the anchorage web
h_b	Steel beam height
h_e	Equivalent viscous damping coefficient
h_{ft}	Height of tension fillet welds
h_{fv}	Height of shearing fillet welds
h_s	Vertical height of the overhang of the vertical stiffener
K_i	Curve stiffness
k	Beam-to-column bending stiffness ratio
k_c	Strength improvement coefficient of concrete under lateral compressive force
k_E	Modulus ratio
k_D	Diameter ratio
k_s	Reduction factor of cross-section
k_y	Coefficient obtained by Samon's test
L_b	Net beam length
L'	Length from the beam end to the column surface
l	Length of stiffening rib
l_a	Anchorage length
l_{ft}	Length of tension fillet welds
l_{fv}	Length of shearing fillet welds
l_p	Distance of vertical stiffener
M	Bending moment
M_{cu}	Ultimate bending capacity of blind bolted joints
M_e	Elastic bending capacity
M_l	Bending moment at left beam ends
M_p	Ultimate bending capacity
M_r	Bending moment at right beam ends
M_T	Ultimate bending capacity when the T-plate fail
M_{uC}	Bending resistance
M_{uE}	Ultimate bending moment at the beam plastic hinge
M_{um}	Simulations ultimate bending capacity
M_{ut}	Tests ultimate bending capacity
m	Coefficients related to the yield strength of steel
N	Axial compressive bearing capacity
N_{bo}	Bolt tension
N_c	Ultimate bearing capacity of CFDST column subjected to axial pressure

N_{fb}	Maximum tensile force of T-plate
N_o	Bearing capacity obtained by simple subposition method
N_t	Ultimate load in tests
N_u	Calculated ultimate bearing capacity
N_C^1	Calculation results obtained by limit equilibrium method of CFST
N_C^2	Calculation results obtained by the method of the Unified Theory of CFST
n	Axial compression ratio
n_c	Index coefficient of compressive damage of concrete
n_t	Index coefficient of tensile damage of concrete
P	Lateral load
P_m	Maximum beam end load
P_j^i	Peak load of the i th cycle when the displacement control is j
P_j^1	Peak load of the first cycle when the displacement control is j
P_{max}	Average maximum quasi-static cyclic load
P_v	Cross-sectional ratio
P_w	Tension in the T-plate flange transferred by fillet weld
P_y	Horizontal yielding load
p	Confining pressure
p_f	Confining pressure from CFRP
p_i	Confining pressure from inner steel tube
p_o	Confining pressure from outer steel tube
p_o'	Confining pressure from outer concrete
$\bar{p}_o$	Confining pressure considering equivalent reduction factor
p_s	Confining pressure from steel tube
r_o	Radius of equivalent circular steel tube
S_{BFDE}	Area in the loop $BFDE$
$S_{j,ini}$	Initial elastic rotational stiffness
S_{OAB}	Shadowed area within OAB
S_{OCD}	Shadowed area within OCD
T	Load time
t	Wall thickness of the steel tube
t_a	Rib thickness
t_b	Thickness of the beam flange
t_{bf}	Thickness of the T-plate web
t_{cf}	Wall thickness of CFRP
t_{eb}	Flange thickness of T-plate
t_{ef}	Flange thickness of steel beam
t_f	Flange thickness of the I-shaped beam
t_i	Wall thickness of inner steel tube
t_o	Thickness of equivalent circular steel tube
t_s	Thickness of square steel tube
t_1	Web thickness of H-shaped steel beam
t_2	Flange thickness of H-shaped steel beam

U	Strain energy
U_c	Strain energy of concrete
U_{si}	Strain energy of inner steel tube
U_{so}	Strain energy of outer steel tube
u	$u = \varepsilon z / \varepsilon \theta$
u_r	Lateral deformation
u_{ri}	Lateral deformation of inner steel tube
u_{ro}	Lateral deformation of outer steel tube
u_z	Vertical deformation
v	Ratio of wall thickness to side length
V_j	Horizontal shear force in panel zone
V_o	Reactive force at the horizontal roller
V_s	Ultimate shear resistance of the steel tube web
V_{uC}	Ultimate shear resistance obtained from calculations
V_{uE}	Ultimate shear resistance obtained from tests
V_v	Ultimate shear resistance of the vertical stiffener
ΔV_c	Volume expansion of whole concrete
ΔV_{co}	Volume expansion of outer concrete
ΔV_{ci}	Volume expansion of inner concrete
ΔV_{cmin}	The minimum value of ΔV_c
W	External work
W_c	External work of concrete
W_{si}	External work of inner steel tube
W_{so}	External work of outer steel tube
w	Coefficient of stiffness restoration
w_c	Weighted coefficient of compressive stiffness restoration
w_t	Weighted coefficient of tensile stiffness restoration
x	Applied displacement
x_l	Number of stiffening ribs
(x_{io}, y_{io})	Initial coordinate of the measuring point
α	Steel ratio of CFST or tension-compression strength ratio
β	Tangent angular changes in the direction of y-axis
β_f	Strength reduction factor of steel tube
σ	Stress
σ_c	Compressive strength
$\bar{\sigma}_c$	Effective compressive stress
σ_{c0}	Peak value of compressive stress
$\sigma_{c\theta}$	Circumferential stress of concrete
σ_{cr}	Radial stress of concrete
σ_{cz}	Axial stress of concrete
σ_r	Radial compressive stress
σ_s	Yield strength
$\sigma_{s\theta}$	Lateral tensile stress
σ_{sz}	Compressive stress under axial compression

$\sigma_{si\theta}$	Circumferential stress of inner steel tube
σ_{sir}	Radial stress of inner steel tube
σ_{siz}	Axial stress of inner steel tube
$\sigma_{so\theta}$	Circumferential stress of outer steel tube
σ_{sor}	Radial stress of outer steel tube
σ_{soz}	Axial stress of outer steel tube
σ_t	Tensile stress
$\bar{\sigma}_t$	Effective tensile stress
σ_{t0}	Peak tensile stress
$\sigma_{12}/\sigma_{23}/\sigma_{13}$	Corresponding normal stresses on the principal shear stress element
$\sigma_1/\sigma_2/\sigma_3$	Principal stress
ε	Strain
$\varepsilon_a, \varepsilon_b, \varepsilon_c$	Strains in horizontal, 45° and vertical directions
$\tilde{\varepsilon}_c^{pl}$	Plastic strain of compressive damaged concrete
$\tilde{\varepsilon}_t^{pl}$	Plastic strain of tensile damaged concrete
ε_{c0}	Strain at the peak compressive stress
ε_r	Radial strain
ε_{ri}	Radial strain of inner steel tube
ε_t	Tensile strain of concrete
ε_{t0}	Strain at the peak tensile stress
ε_θ	Circumferential strain
ε_z	Axial strain
ε_{sc}^p	Nominal composite proportional limit strain of CFST column
ε_{scc}^p	Proportional limit strain of CFDST column
ε_{oc}^{el}	Elastic strain of compressive concrete
ε_{ot}^{el}	Elastic strain of tensile concrete
γ	Shear strain
γ_{xy}	Shear strain at any speckle
γ_p	Peak shear strain
γ_μ	Concrete strength reduction factor
θ	Beam-to-column rotation
θ_e	Beam-to-column relative rotation corresponding to M_e
θ_t	Angle value at moment t
θ_{to}	Angle value at the initial moment
δ_n	Lateral displacement of column top
δ_m	Lateral displacement of joint central point
τ	Shear stress
τ_{bond}	Average bond shear stress
τ_p	Peak shear stress
τ_s	Shear stress
$\tau_{12}/\tau_{23}/\tau_{13}$	Principal shear stress
λ_i	$\lambda_i = D_i/2t_i$
λ_j	Bearing capacity reduction coefficient

λ_o	$\lambda_o = D_o/2t_o$
μ	Ductility coefficient
μ_f	Coefficient of friction
μ_m	Ductility ratio at the maximum load
μ_μ	Ductility ratio at the collapse point
ξ	Confinement coefficient
ξ_{ssc}	Equivalent confinement coefficient of CFDST column
ζ	Equivalent reduction factor
Π	Potential energy
Π_c	Potential energy of concrete
Π_{si}	Potential energy of inner steel tube
Π_{so}	Potential energy of outer steel tube
Δ	Lateral displacement
Δ_j^i	Beam end displacement of the i th cycle when the displacement control is j
Δ_m	Displacement corresponding to the maximum load
Δ_y	Nominal yielding displacement
Δ_μ	Collapse displacement
$\Delta_4, \Delta_5, \Delta_6, \Delta_7$	Displacements of the angular points

Acronyms

ACI	American Concrete Institute
ATC	Applied Technology Council
AIJ	Architectural Institute of Japan
AISC	American Institute of Steel Construction
ASCE	American Society of Civil Engineers
BS	British Standards
CCD	Charge coupled device
CDP	Concrete damage plastic
CFST	Concrete-filled steel tube
CECS	China Association for Engineering Construction Standardization
CFRP	Carbon fiber reinforced polymer
CFDST	Concrete-filled double steel tubes
DSCM	Digital speckle correlation method
FE	Finite element
FEA	Finite element analysis
FEM	Finite element modeling
IMP	Image manipulation program
LRFD	Load and resistance factor design
MTS	Machine tractor station
RC	Reinforced concrete
SRC	Steel reinforced concrete
TUST	Twin-shear unified strength theory

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Chapter 1

Introductions

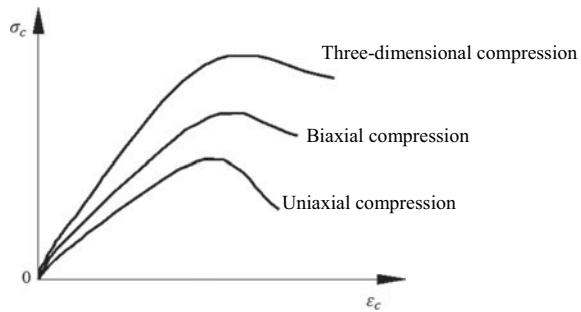


Abstract This chapter reviews the development of Concrete-Filled Steel Tube (CFST) including the beam-to-column connection types in CFST moment frames. As CFST has been used widely in high-rise buildings and long-span bridges, it has been reinforced into some new structural forms. Four types of reinforced CFST are presented according to different cross-sections and confining materials, and concrete-filled double steel tubes (CFDST) is the effective form of reinforced CFSTs to reduce the cross-section area and improve structure performances.

1.1 Principles of Concrete-Filled Steel Tube (CFST)

When concrete is poured into a steel tube, as shown in Fig. 1.1, the lateral deformation of concrete is limited by the steel tube. Almost 100 years ago, Richart et al. (1928) concluded that the confined concrete has higher axial compressive strength, stiffness and deformation capacity. The so-called confined concrete is in a complex three-dimensional compressive state due to confining effect from the steel tube. Based on a lot of analyses and summaries of theoretical researches and tests, the effect of lateral restraint on the stress-strain curve of compressed concrete is presented in Fig. 1.2. For the concrete, more obvious lateral restraint is, the more concrete strength is improved, so the confining effect of CFST under axial compression is the fundamental reason for special performances of CFST. When the composite column is close to failure, the contribution of the steel tube is mainly reflected in the constraint on concrete, and this constraint can limit concrete to its ultimate deformation. The steel tube can restrain expansion and extension of micro cracks in the core concrete, thus compressive strength of concrete is improved. Meanwhile, the local buckling of the steel is delayed because it can only buckle outwards. Therefore, in the CFST column, the steel tube encloses the core concrete and is used as both longitudinal and lateral reinforcements. This mechanism can guarantee the working continuity of concrete under high axial load, and brittleness of plain concrete is changed qualitatively.

Under the condition that the steel tube can effectively restrain lateral expansion of concrete, increasing yield strength of the steel tube cannot effectively improve bearing capacity of the column; the strength improvement of core concrete is the

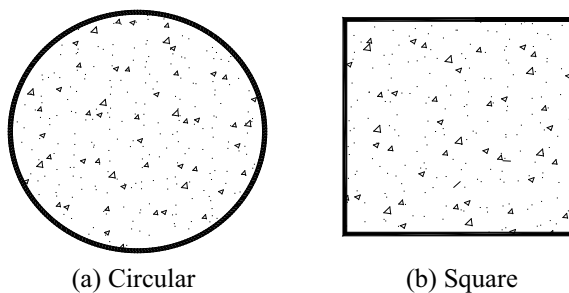
Fig. 1.1 CFST**Fig. 1.2** Stress-strain relationships of concrete

decisive factor of bearing capacity for CFST (Cai 2003; Han and Yang 2001; Zhong 1996). The bearing capacity of the CFST column is much higher than the sum of the respective bearing capacity of concrete and steel tube. The main benefit of using CFST is that it utilizes the advantages of both steel and concrete, viz. steel members have high tensile strength and ductility, while concrete members are advantageous in compressive strength and stiffness (Kenji et al. 2004; Kuranovas et al. 2009; Schneider 1998). It provides not only an increase in the load carrying capacity but also rapid construction, and thus additional cost saving (Roeder et al. 1999; Zhong 1999).

1.2 Applications and Developments of CFST

CFST started from the circular cross-section, since the circular tube is easy to construct and has good confinement to the infilled concrete. It often acts as one of the fundamental bridge types, which is widely applied in China for its extremely high aesthetic value (Alfawakhiri 1997; Han et al. 2014; Roeder et al. 2010). Since the CFST has significant advantages in section modulus, stability, ductility and better damping characteristics and fire resistance etc., the use of CFST columns in different

Fig. 1.3 Two traditional cross-section types of CFST



areas of constructions has become an attractive solution in the past several decades. In comparison to more traditional construction methods, circular and square CFSTs, as shown in Fig. 1.3, have been applied widely in civil engineering. The CFST enables faster construction, as the steel tube acts as a permanent formwork facilitating the pouring of concrete without the need of a formwork removal. Moreover, using confinement in the form of a square steel tube can greatly improve the ductility of normal or high-strength concrete (Cai 2003; Zhong 1996).

Due to the small section dimension, high compressive strength, large stiffness, and excellent deformation capacity, the use of CFST piers is attractive, especially to high-pier and super-high-pier bridges located in mountains. Since the late of 1980's, two new application fields of CFST were opened up in China, which are highway arch bridges or urban arch bridges and high-rise or super high-rise buildings. The CFST is used in many structural applications including columns supporting platforms of offshore structures, roofs of storage tanks, bridge piers, piles, and columns in seismic zones, as shown in Fig. 1.4. Circular CFST often acts as one of the fundamental bridge types, widely applied in China for its extremely high aesthetic value, as shown in Fig. 1.4c. The CFST has been widely used in the industry due to the numerous advantages they offer over conventional purely steel or purely reinforced concrete structural elements. Especially, CFST supports, as a new support form, have been gradually applied in underground roadway, as shown in Fig. 1.4d. Problems of soft rock roadway supporting are difficult in many mining fields in China, and it is one of the most important issues that restricts the coal mining safety and effectiveness. The invention of CFST supports opened up a new way to solve the problems of soft rock roadway. The ultimate bearing capacity of CFST supports is greater than the sum of the ultimate bearing capacity of the same amount of steel column and plain concrete column.

1.3 Researches on CFST

Experimental studies on CFSTs have been on-going for many decades. This chapter reviews the development of the family of CFST structures to date and draws a research framework on CFST members, such as static performances, seismic behaviors and

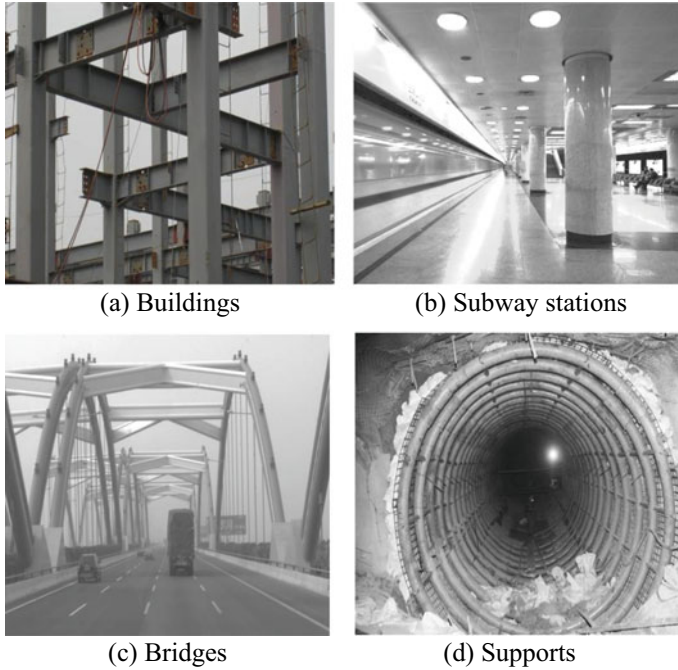


Fig. 1.4 Applications of CFST

fire resistance (Han et al. 2003, 2005; Tu et al. 2014; Yang et al. 2009). Furthermore, some researches on bond performances between steel tubes and concrete, and shrinkage and creep of CFSTs also have been carried out (Baltay and Gjelsvik 1990; Morishita et al. 1979; Shanmugam and Lakshmi 2001). The development of research on CFST structural members in most recent years, particularly in China, is summarized and discussed as following.

Axial compression tests and eccentric compression tests were carried out to analyze the mechanical performances, and the effects of length-diameter ratio and confinement coefficient on the mechanical performances were studied. The strength calculation was discussed by using the plastic hinge theory and crushing theory respectively. The formulas for calculating the bearing capacity and the ultimate load were put forward. Some shear tests and finite element simulations on composite columns were carried out, and analysis parameters include thickness of the steel tube wall, shear span ratio, concrete strength and axial compression ratio.

The stability analysis of the column was conducted based on the concrete characteristics and stress distribution in steel tubes. The constitutive relationship between the steel tube and concrete was established, then formulas for calculating critical load and stable load were put forward.

The seismic behaviors of columns under compression-bending and reciprocating shear forces were also studied. The effects of axial compression ratio, width-thickness

ratio and concrete strength were analyzed. The results showed that specimens had a good local buckling resistance due to the confinement effect on concrete. Compared with reinforced concrete columns, the CFST columns have better energy dissipation ability, and the strength degradation became smaller in the reciprocating load process.

1.4 Connections in CFST Structures

With the increasing number of applications of CFST columns, safe and economical designs for beam-to-column connections are required. The most important part of a frame structure is the beam-to-column connection. Many joint types have been developed in CFST moment frames, and adequate structural performance was demonstrated under seismic loading conditions. The common joint types, which have been widely adopted, can be divided into through-beam joint, exterior stiffening ring joint (external diaphragm joint), diaphragm-through joint, vertical stiffener joint, welded haunch joint, anchored T-plate joint, bolted joint, as shown in Fig. 1.5.

1.4.1 *Through-Beam Joint and Through-Column Joint*

Sometimes, the beam-to-column connection can be classified as two types according to connection details, namely, through-column and through-beam connections. For through-beam connections shown in Fig. 1.5a, the beam or its reinforcements directly pass through the panel zone, then other embedded elements are used to enhance the connection. The through-column connections utilize stiffeners to connect the RC beams to the CFST columns, and these connections may cause difficulties in field construction due to the associated excessive material and labor cost, because the complicated fabrication, welding in situ, or hole drilling on site cannot be avoided. Test results showed that the through-beam connections had better seismic performance than the through-column connections, as reported in the literature (Bai et al. 2008; Nie et al. 2008; Schneider and Alostaz 1998). Besides, there is another similar through-beam connection called continuous web plate joint, which is recommended by CECS 28-90. The web of the steel beam passes through the steel tube wall and the concrete inside the steel tube, and the beam flange is connected (welded) to the steel tube wall. It is generally believed that it is convenient to pour concrete in the tube. Meanwhile, most of the shear force is transferred from the continuous web plate to the core concrete, and the additional stress on the steel tube is small and the stiffness is large.

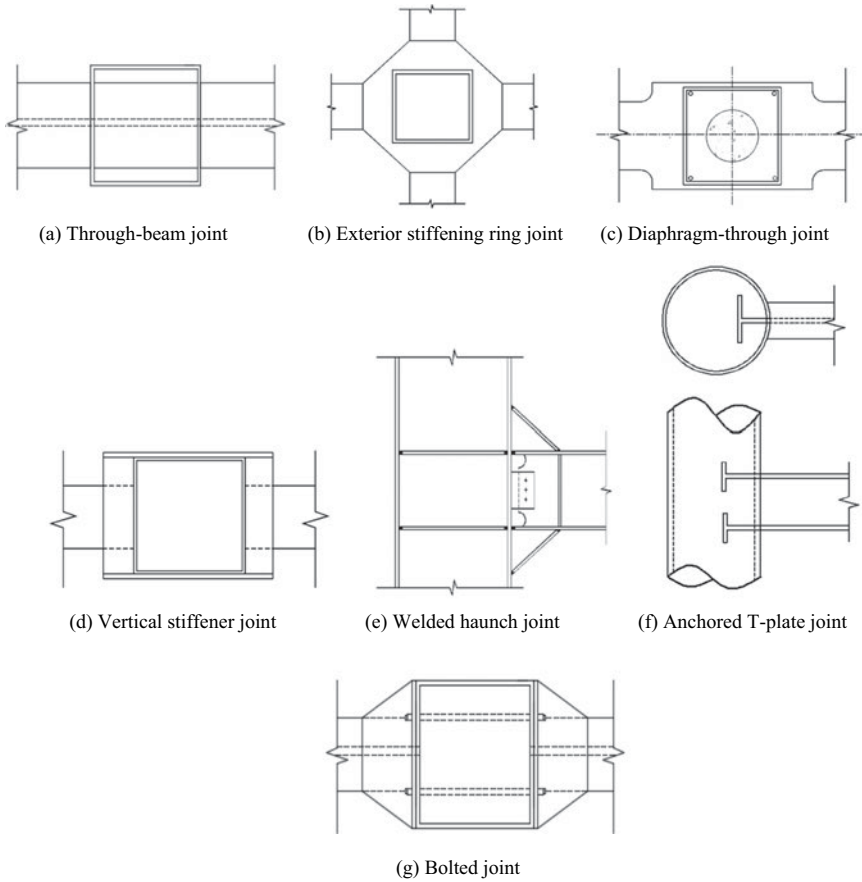


Fig. 1.5 Common joint types

1.4.2 Exterior Stiffening Ring Joint (External Diaphragm Joint)

The connection with external stiffening ring is a typical rigid joint for a CFST column to connect a steel beam or a RC beam. The stiffening ring is located outside the CFST column to connect the adjacent beam to transfer the bending moment and shear force at the beam-to-column junction, as shown in Fig. 1.5b. The additional stiffening ring can significantly increase the beam-to-column rotation stiffness, and the stiffening ring is considered as a main member to affect mechanical properties of the connections. While for the connection between a CFST column and a steel beam, it is often called external diaphragm joint. The upper and bottom external diaphragms as same as additional stiffening rings, transfer internal shear force and bending moment from the steel beam effectively. External stiffening ring joints have also been investigated

by a large number of researchers, including to connect steel beams or RC beams. This type of joint is recommended by the existing codes for CFST structures, such as AIJ standard for structural calculation (2001) and *Chinese technical codes* (CECS-159 2004; *The Chinese Code for Concrete Filled Steel Tubular Structures* GB50936-2014) for tubular columns, because of its large stiffness, good plasticity, and high bearing capacity.

1.4.3 Diaphragm-Through Joint

The diaphragm-through joint is commonly applied to connect steel beam, as shown in Fig. 1.5c, and the difference from external diaphragm joints are shown in Fig. 1.6. In diaphragm-through joint, the steel tube is disconnected and two diaphragms are horizontally placed at the upper and lower flange of the steel beam. Sometimes the through-diaphragm is not chosen as an extended form out of the column, so the type is called as the interior diaphragm joint. Both interior diaphragm joints and diaphragm-through joints are rigid beam-to-column connections that are used in many applications. The interior diaphragm is perforated for pouring concrete, thus large column cross-section size is more suitable. The plastic zone of the tube appears around the intersections between the tube and the diaphragm, while the plastic zone of the diaphragm appears along the cross-section lines enclosed by the steel tube. Besides, the influence of steel beam web on the plastic zone of the steel tube is significant and cannot be neglected.

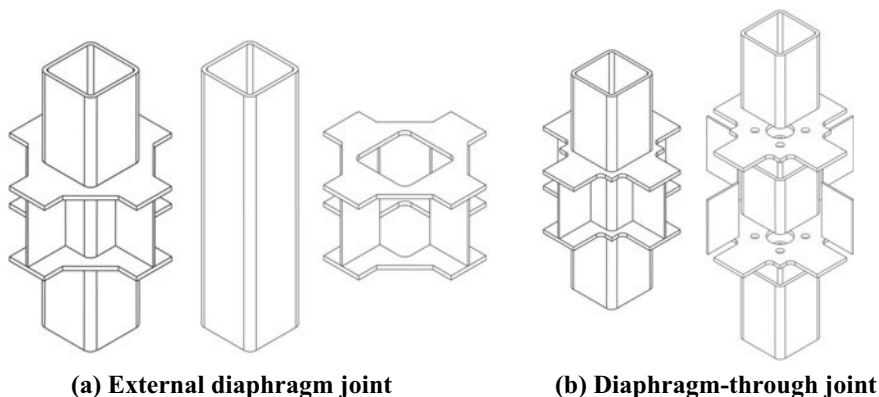


Fig. 1.6 Comparison of typical joints to steel beams

1.4.4 Vertical Stiffener Joint

In vertical stiffener joints, the steel plate is placed vertically as the stiffener to connect the steel beam flange and the column. The vertical stiffener is groove welded on the column flange, and its extension is welded to the end plate sides, so the extension is the same long as the widened end plate, as shown in Fig. 1.5d. End plates connect with the upper and lower flanges of steel beams, thus the construction is simple and reasonable. The vertical stiffener joint has the advantage of transferring load reliably thus it has a wider prospect in applications. Vertical stiffener connection composed of transverse and vertical steel plates can be applied in high-rise residential buildings (Xiong et al. 2019; Zhang et al. 2018). The variables studied in these experiments include the joint type, axial compression ratio, cross-sectional area and width-to-thickness ratio of the vertical stiffener, and presence of concrete. The results indicate that the cross-sectional area of the vertical stiffeners plays a critical role in the performance of the panel zone.

1.4.5 Welded Haunch Joint

The haunch retrofitting technique has been employed, primarily to stiffen the beam-to-column connections that controls the hierarchy of strength within the beam and column members. It also avoids shear hinges in RC or steel frame structures subjected to earthquake imposed lateral loads. As shown in Fig. 1.5e, the steel plates of upper and bottom haunches are inclined in some depth. In consideration of large moment transferred by the haunch connection to the CFST column, the column web often needs to be stiffened by a steel plate. The connection with haunches is easy to meet the requirement of the code for seismic design of buildings, and the haunch joint would be a better solution if enhancement of both positive and negative moments were required. So, haunches are widely applied in designs of seismic moment connections to avoid brittle fractures since haunches are effective to keep plastic areas away from column flanges (Kurejková and Wald 2017; Saberi et al. 2019; Tanaka 2003). The increase in moment capacities due to the haunch is also found to be substantial. The haunch joint exhibits good plastic deformation in the loading process, which may help the plastic hinge move to outside of the column. The characteristic of this joint type is more predictable and reliable when the frame is subjected to reverse load arising from wind or earthquake.

1.4.6 Anchored T-Plate Joint

The anchored T-plate joint is to connect a CFST column and an H-section steel beam. T-plates are used as the embedded elements in the joint core, as shown in Fig. 1.5f.

The length and the location of anchored T-plates depend on the size of the CFST column in consideration of the pouring of concrete. There are two types of anchored T-plate joints according to the directions of webs of the anchored T-plates. One is: the web of T-plate is upright as the top one in Fig. 1.5f. The T-plate web is welded inside the steel tube to connect the steel beam web. The other is: the webs of T-plate are horizontal as the bottom one in Fig. 1.5f. The T-plate web is welded inside the steel tube to connect the steel beam flange. The vertical T-plate web is beneficial for concrete to be poured into the steel tube, and the anchored T-plate can be chosen a longer one, while the horizontal T-plate web has the advantages of direct force-transferring, large stiffness, and small deformation. Basically, test results (Li 2007) showed that the connections exhibit good static and seismic behaviors, and with a proper design, the anchored T-plate joint can be reliably used in practical engineering applications.

1.4.7 Bolted Joint

Most rigid connections require considerable in situ welding or embedded components to achieve sufficient moment capacity. Moreover, the welded beam-to-column connections were found to have brittle fractures in structures during the post-earthquake survey. So the connections using high-strength bolts have been used in CFST structures as alternative solutions. The bolted connection with end-plates avoids the potential failure associated with weld fractures and requires only shop welding and on-site bolting works. In this joint system, through-bolts and end-plates (or T-plates) are force transfer members. As shown in Fig. 1.5g, T-plates along with end plates connect with the steel tube in the joint zone, and high-strength bolts connect end plates together with the steel tube before pouring concrete. Meanwhile, T-plate webs connect with steel beam flanges by using standard high-strength bolts. The quasi-static test and FE Modeling were carried out by many researchers (Beena et al. 2017; Jiang et al. 2019a, b; Piluso and Riaaano 2019a; Zhang et al. 2020), and results showed that the bolted joints had many good mechanical behaviors such as large bearing capacity, good ductility, and strong initial rotational stiffness. Because the bolted joint with T-plates is a typical semi-rigid connection form (Waqas et al. 2019), it has great deformation ability and better seismic performance.

1.5 Reinforced CFSTs

As CFST has been used widely in high-rise buildings and long-span bridges, the columns are required to have higher bearing capacity and better ductility, especially subjected to more and more axial loads. Since traditional CFST only has single steel tube to confine concrete, it has been reinforced into some new structural forms. Reinforced CFST means that besides the outmost steel tube, there is another steel

tube, steel column or carbon fiber reinforced polymer (CFRP) sheet confining the concrete (Wang et al. 2005a). These heavy-loaded columns have been studied by quite a number of researchers and have been verified its feasibility in theoretical researches and engineering practices. The advantages of reinforced CFSTs include:

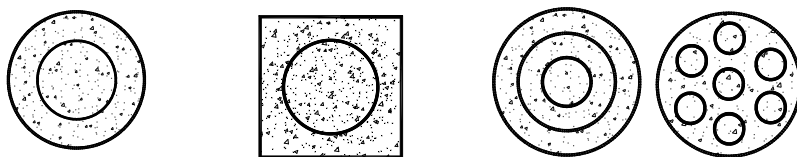
- (1) Improve fire resistance of CFST. CFSTs need more fire protection due to the steel tubes, while in reinforced CFST, the other confining material such as another steel tube, steel column or CFRP sheet can provide more restraining force for the infilled concrete and provide vertical and lateral resistance if the outer tube could be damaged in the blaze.
- (2) Strengthen the existing CFST. The emergency capacity including strength and stiffness is upgraded, so reinforced CFST structures can bear a high earthquake intensity and terrorist attacks.
- (3) Apply thin-walled tubes in fat columns as well. Thick-walled tubes are difficult to fabricate and the confining effects are not evident in CFST structures, and thin-walled tubes can achieve the ideal stress state. For reinforced CFSTs, the application of other confining materials can reduce wall thickness of the large diameter steel tube and consumption of high strength steel under the condition of guaranteeing the same performances.

Furthermore, this column, without needing to consider the limit of axial compression force ratio, can reduce the column size to provide substantial benefits where floor space is at a premium such as in car parks and office blocks. Therefore, reinforced CFST has the advantages of larger bearing capacity, greater plasticity, upstanding toughness and good fire resistance. It could be used in industrial buildings, high-rise and super high-rise buildings, bridges and underground structures. There are four types of reinforced CFST structures concluded in this monograph. I: CFST reinforced by inner steel tube; II: CFST reinforced by inner section steel; III: CFRP-confined CFST; IV: hollow CFDST. Cross-section types $a \sim m$ of reinforced CFSTs are illustrated as follows.

1.5.1 I: CFST Reinforced by Inner Steel Tube (Including CFDST)

Researches (Roeder et al. 2010) showed that the circular steel tube provided greater bond stress transfer, better confinement, and greater shear resistance to the in-filled concrete, so the CFST column reinforced by inner circular steel tube belongs to the most effective cross-section type. Four cross-section types of CFST columns reinforced by inner steel tube columns are shown in Fig. 1.7, where Types a , b show the cross-sections of concrete-filled double steel tubes (CFDST), which consists of an inner steel tube and an outer steel tube, both filled with concrete. Figure 1.7c shows the cross-section of circular CFST reinforced with multi-tube.

The bearing capacity of stub columns of Types a , c was analyzed by the superposition principle of limit equilibrium theory (Cai 2003). Test verification suggests that it



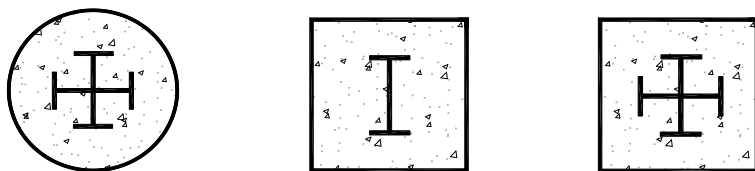
(a) Inner circular-outer circular CFST (b) Inner circular-outer square CFST (c) Circular CFST reinforced with multi-tube

Fig. 1.7 CFST reinforced by inner steel tube

is better to use a concentric circular steel tube inside the CFST column in application, but setting more concentric steel tubes has a little benefit for the bearing capacity. Experiments on CFDST columns of Type *b* were carried out by Zhang et al. (2013). Working mechanism of concrete and the confining effect of inner tube were considered in the calculation of axial bearing capacity. Comparison between test results and calculations showed that the common method of CFST column was not suitable for the CFDST column. And test results confirmed that CFDST columns exhibited better seismic resistance and higher strength, particularly when they were subjected to high axial compressive loads. CFDST columns can also effectively reduce the cross-section area and significantly improve the fire resistance.

1.5.2 II: CFST Reinforced by Inner Section Steel

Through investigating the damages caused by earthquakes, people have realized the design of columns plays a key role in preventing the collapse of building in strong earthquakes. In particular, with the increase of height and span of building, columns have to bear higher and higher axial load. Based on a series of research achievements for the steel reinforced concrete (SRC) column, CFST reinforced by inner I-steel or cross steel was studied and applied. The main cross-section types are shown in Fig. 1.8. In these composite columns, structural section steel is inserted into the steel tube and self-consolidating high-strength concrete is filled into the void between them. CFST reinforced by inner section steel can reduce the cross-sectional size and improve the stability of the column. Test and theoretical investigations (Wang et al. 2005b; Zhao 2003) show that the behavior of self-consolidating columns and



(d) Circular CFST with cross steel (e) Square CFST with I-steel (f) Square CFST with cross steel

Fig. 1.8 CFST reinforced by inner section steel

vibrated columns are almost the same; the encased structural section steel effectively delays or restrains the generation of shear sliding crack in high-strength concrete and therefore the ductility of the columns is improved; concrete strength, width-to-thickness ratio and the area of encased structural section steel have significant effects on the bearing-capacity and ductility of the columns. Results also show the composite column has higher ductility and higher vertical residual load-carrying capacity after failure. These features of the composite column will prevent the collapse of buildings in strong earthquakes. So this type of composite column can be used in structures of strong seismic intensity zone.

Especially, in some engineering applications, section steel columns need to be repaired and strengthened. This column type is used to reinforce the internal steel column by setting an external steel tube and pouring concrete. It is an easy and applicable way to set steel tubes outside the steel column and pour concrete in tubes. Formwork and stirrups are not required, and it is more convenient for construction.

1.5.3 III: CFRP-Confined CFST

CFRP has been widely used to repair and retrofit of deficient structures in recent decades, because externally bonded CFRP material in the form of sheets or plates is particularly well suited for flexure and shear (Chaalal and Shahawy 2000; Che et al. 2012; Xiao 2004; Zheng et al. 2018). Due to high tensile strength of CFRP, it provides circumferential confining force as the steel tube to improve axial compressive bearing capacity of core concrete. As discussed by Gu and Li (2011), Tao et al. (2005) and Wang et al. (2016, 2017), most of the research conducted has focused on the use of CFRP for CFST structures. Carbon fiber sheets or plates are attached to the steel tube or concrete in a CFST member to increase its bearing capacity and ductility. It can be concluded that the ultimate lateral strength and flexural stiffness of CFRP-confined CFST columns increase with the increasing number of CFRP layers. Meanwhile, the ductility of specimens increases slightly with the number of CFRP layers. The CFRP cylinder can also impede buckling of the stub column, leading to dramatic improvements in buckling and post buckling behavior of the entire system.

Three types of CFRP-confined CFST columns are considered with different CFRP confinement including outer circular CFRP, inner circular CFRP and outer square CFRP, as shown in Fig. 1.9. The CFRP cylinder is wrapped outside the circular CFST column in Fig. 1.9g; the CFRP cylinder is placed inside the square CFST in Fig. 1.9h and the CFRP cylinder is wrapped outside the square CFST column in Fig. 1.9i. As can be seen, steel tubes together with the confined concrete can resist the axial compression remarkably, while the CFRP cylinders can provide the lateral confinement to the steel tube or concrete directly and complementary action between steel tube and concrete is strengthened through the higher confinement of CFRP.

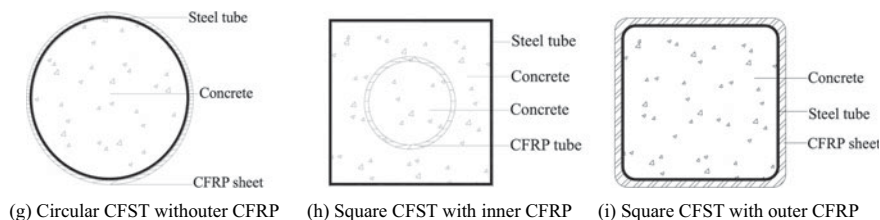


Fig. 1.9 CFRP-confined CFST

1.5.4 IV: Hollow CFDST

In the present study, concrete-filled double skin steel tubes are lighter and more cost-effective compared to CFST. It is a composite member containing two steel tubes with concrete filling the space between them, so it is called hollow CFDST. The core concrete in CFDST is removed in order to lighten the weight and retrofit seismic behavior. Research studies related to hollow CFDST have been extensively undertaken. For instance, Tao and Han (2006) conducted an experimental research on beams, columns, and beams-columns with various cross-sections. A series of tests on hollow CFDST stub columns and beam-columns were implemented by Han et al. (2006, 2010, 2011), Huang et al. (2013), Tao et al. (2004), and Wang et al. (2015). Regarding analytical studies, Pagoulathou et al. (2014) studied the behavior of CFDST stub columns subjected to concentric axial compression loads, and then he proposed a new formula to evaluate the strength of hollow CFDST in accordance with EC4 (2004). Hassanein et al. (2015) investigated the compressive behavior of hollow CFDST with various cross-sections. Recently, Liang (2018) suggested a mathematical model for the simulation of the performance of slender hollow CFDST columns with high-strength concrete subjected to eccentric loads. The cross-sections of hollow CFDST are shown in Fig. 1.10. When structures are subjected to strong wind and earthquake action, the vertical members are required to have large bending stiffness and seismic resistance. And all above FEM and experiment results indicated that hollow CFDST has sufficient strength, plasticity, and toughness through the reasonable design of diameter and thickness of the inner and outer tubes.

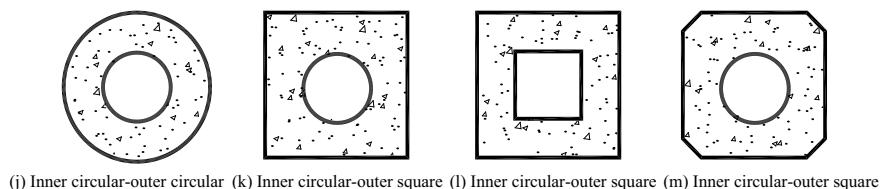


Fig. 1.10 Hollow CFDST

Fig. 1.11 Applications in the government building, Germany



1.5.5 Applications of CFDST

The CFDST structure has been verified its feasibility in theoretical researches and engineering practices, and it exhibits excellent structural and constructional benefits, showing a great potential for more development. So in this monograph, CFDST is mainly studied including the column performance and joint construction. With stronger confinement effects, the CFDST column exhibits higher ultimate bearing capacity and better seismic performance than the common CFST column. It also has a convenient construction process, remarkable economic benefits, so there is a broad application prospect in civil engineering.

The principal cross-sections of the CFDST column are the inner circular-outer circular (Type *a*) and inner circular-outer square (Type *b*). The CFDST column with double circular steel tubes (Type *a*) was applied in the construction of city hall in Wuppertal, Germany, as shown in Fig. 1.11. The column with no more than 600 mm in diameter resolved the problem of one fire-resisting and overloading column with the capacity of 8000kN load. As a design solution, 8 colossal pillars 530 meters high once were assumed in East Tower (Fig. 1.12), Guangzhou, China. In this scheme, the CFDST column was considered to bear super large axial force.

1.6 Monograph Organization

This monograph is divided into ten chapters. It begins with an introduction, in this chapter. Chapter 2 first studies the behavior of stub columns under axial compressive load and the typical failure mode of the composite column is summarized. The ultimate bearing capacity of reinforced CFST columns is derived by the unified strength theory. Meanwhile, a model is proposed to analyze the optimal design of the inner tube. Two methods including the Unified Theory of CFST and an elastoplastic limit equilibrium method have been applied to calculate the axial compressive strength of

Fig. 1.12 A design solution in East Tower, China



four types of reinforced CFSTs. CFDST is testified to be the more applicable CFST structure by comparison of mechanical properties and steel ratio, and the optimal section design of CFDST is predicted.

Chapter 3 reviews and compares different design expressions for CFDST members in current specifications. The parameters affecting the compressive stiffness of CFDST columns are summarized and improved models for evaluating the stiffness, parametric analysis are proposed. The axial stiffness can be obtained by the equivalent confinement coefficient. Using the example of hollow CFST, the calculation results are testified by the energy variational method.

Chapter 4 introduces five types of beam-to-column connections suitable for CFDST columns. To research a reasonable form of beam-to-column connection is the top priority in CFDST frame structures. The constructional detail and force transfer mechanism of each beam-to-column connection are simply analyzed. The connection design should take full advantage of the characteristic of double tubes in the CFDST structure, so a series of construction suggestions for joint design parameters are proposed to improve the seismic performance.

Chapter 5 introduces ring beam joints with discontinuous outer tube between CFDST columns and RC beams, which can be considered as a typical through-beam connection. The seismic behaviors of the joint are studied including the load-deflection performance, typical failure modes, stress and strain distributions, and energy dissipation capacity. More consideration was focused on parametric analysis for seismic resistance through software ANSYS.

Chapter 6 focuses on the structure form, mechanical characteristics and force transfer mechanism of the vertical stiffener connection to steel beams. The differences of bearing capacity, stiffness, ductility, energy dissipation and stiffness degradation are summarized between CFDST columns and the square CFST column. Besides, a

finite element model is established to investigate the effects of different parameters on mechanical performances. Analytical models of shear force and bending moment are established through the appropriate material constitutive equations and equilibrium theory.

Chapter 7 presents the quasi-static tests of ten external diaphragm joint specimens with various parameters. The failure modes, failure mechanism, hysteretic behaviors, skeleton curves, ductility, energy dissipation capacity and stiffness degradation of the external diaphragm joints are summarized. The differences of each mechanical behavior among all the specimens are showed, and the effects of parameters change can reflect initial defects of this joint type in practical engineering and provide useful solutions.

Chapter 8 presents four anchored web joints between CFDST columns and H-shaped steel beams tested under the cyclic horizontal load. Test results indicate that the joint specimens have strong connection stiffness, high bearing capacity, significant energy dissipation. In addition, the nonlinear finite element (FE) models are established using ABAQUS to verify those test results and a further parametric investigation.

Chapter 9 presents cyclic loading tests and numerical simulations on six bolted joints to evaluate the connection performance. Hysteresis loops, failure modes, and ultimate bending capacity obtained by experiment research are in good agreement with the FE analysis. The force transfer mechanism of blind bolted joints is very different from the through-bolted joint. The theoretical calculation and parameter influences of the ultimate bending capacity of blind bolted joints are also reported.

Chapter 10 presents the main research conclusions about CFDST structures, and the relevant study for the future work is also discussed.

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Chapter 2

Analysis of Axial Bearing Capacity



Abstract In the reinforced CFST, there are more than one steel tube to confine the concrete, so its compressive behavior is different from the ordinary CFST. Compressive strength is an important parameter for structural members, and most of the researches were concentrated on the superposition method to calculate the ultimate compressive strength (Zhang et al. in *J Comput Theor Nanos* 13(2):1422–1425, 2016), so different formulas were deduced for every different cross-section of reinforced CFSTs. To facilitate the practical application of the composite column in seismic retrofit, the ultimate compressive strength of this reinforced column needs to be accurately calculated and properly modeled. In this chapter, some tests on stub CFDST columns are introduced firstly to testify the advantages of reinforced CFST columns. Then, two models are applied to evaluate the ultimate bearing capacity for various sections of reinforced CFST columns, based on the Unified Theory of CFST and elastoplastic limit equilibrium theory respectively. In comparison with other empirical models, the analysis on bearing capacity is extended to predict the optimal design of reinforced CFSTs. Thus, CFDST is confirmed as the applicable heavy-loading column type, and the optimal cross-section is inner circular-outer square, and a parametric investigation is carried out for inner circular tubes.

2.1 Tests and Phenomena

Four CFDST columns were tested (Zhang et al. 2013), and specimens before and after casting are shown in Fig. 2.1 and Fig. 2.2 respectively. The parameters of steel tubes of all specimens are shown in Table 2.1. The aspect ratio of all the specimens is 3. The concrete was cast in the lab while the steel tubes were provided by the manufacturer. All the specimens were cured under standard conditions in the lab until the concrete design strength was achieved. The compressive strength of the concrete was determined by using concrete cubes with side length of 150 mm. Each time when pouring concrete, concrete cubes were prepared and cured under the same conditions in order to obtain reliable material properties that are shown in Table 2.1.

Fig. 2.1 Specimens before pouring concrete



Fig. 2.2 Specimens after pouring concrete



Table 2.1 Experimental parameters and calculated results

Specimen	Outer tube $B \times t$	f_{so} (MPa)	Inner tube $\Phi D_i \times t_i$	f_{si} (MPa)	Numbers of specimens	Ultimate load N_t (kN)
G1-1	120mm $\times$ 2.6mm	407.5	–	–	1	896
G1-2	120mm $\times$ 2.6mm	407.5	$\Phi 58.5\text{mm} \times 1.4\text{mm}$	352.5	2	980
G1-3	120mm $\times$ 2.6mm	407.5	$\Phi 74.0\text{mm} \times 0.9\text{mm}$	680.0	3	1040
G1-4	120mm $\times$ 2.6mm	407.5	$\Phi 83.0\text{mm} \times 0.9\text{mm}$	597.0	2	1080

The experiments were conducted under a servo hydraulic machine with a capacity of 5000 kN in static loading. The strain gauges for steel were preset before casting the concrete to measure the vertical and circumferential strain of the tubes. The axial deformation was measured by an electronic extensometer. For acquiring the experimental stress-strain curves of the columns, the specimens were tested to failure under monotonically increasing concentric loads, and the load control mode was 0.15 MPa/s in stress at the beginning, then switched to 0.001/s in strain after exceeding 80 % of the theoretical peak compressive strength.

During the compressive pressure loading process, yielding of the outer steel tube and the inner steel tube can determine the final failure of the composite column. The ultimate failure modes generally include bulging failure and shearing failure. In the test, the typical failure mode of the CFDST column was a local failure mechanism as shown in Fig. 2.3. It can be seen that the outer tube behaves the same way of outward bulge as the pure square CFST column. The buckling of stub column is mainly caused by the expansion of concrete under axial load. For the specimen G1-4, the transverse strain of inner and outer tubes at the ultimate state, were $1500 \mu\epsilon$ and $1200 \mu\epsilon$ respectively. Therefore, both outer square tube and inner circular tube provided some confinement to concrete when the column was compressed. Outer concrete with the reinforcements from outer square steel tube also surrounds the inner circular steel tube, so the buckling of the inner steel tube can be postponed. Meanwhile, the inner concrete is under the circumferential confining force coming

Fig. 2.3 Typical failure mode



from both inner steel tube and outer one. With the load increasing, deformation of CFDST columns increased, and the interaction forces between the steel tubes and concrete became stronger and stronger to make the concrete under three-directional compression. The outer steel tube yielded before the inner steel tube. During the ultimate state, concrete was in the plastic state and continually developed micro-cracks inside until the destruction of the specimen. Therefore, the CFDST structure, one type of reinforced CFSTs, has been verified its feasibility in theoretical researches and engineering practices, showing a great potential for more development.

2.2 Calculation by Limit Equilibrium Method

2.2.1 Limit Equilibrium Method

The reinforced CFST column can be divided into different components, and every part would be conducted in a elastoplastic analysis respectively. In the state of limit equilibrium, the ultimate axial bearing capacity of the composite column is composed of the contributions from every member. Take the example of CFDST with inner circular-outer circular (*Type a* in Fig. 1.7), the composite column can be separated as outer steel, inner steel, outer concrete, and inner concrete. Hence, the ultimate bearing capacity includes four parts, i.e.

$$N_C^1 = \bar{f}_{so}A_{so} + \bar{f}_{si}A_{si} + \bar{f}_{co}A_{co} + \bar{f}_{ci}A_{ci} \quad (2.1)$$

where f is defined as compressive strength, and A is defined as the area of cross-section. The subscripts s , c , o and i in this chapter are the abbreviation of *steel*, *concrete*, *outer* and *inner* respectively. And ‘—’ means the actual ultimate strength when the column failed under axial loading.

2.2.2 Unified Strength Theory

Under the ultimate state of the reinforced CFST column, every part can be analyzed by twin-shear unified strength theory (TUST). The TUST (Yu 2004) considers two larger principal shear stresses and the corresponding normal stresses and their different effects on the failure of materials. When the relationship function between them reaches one ultimate value, the material can be defined as failure at this state which is formulated as follows

$$F = \tau_{13} + b\tau_{12} + \beta(\sigma_{13} + b\sigma_{12}) = C \quad \text{when} \quad \tau_{12} + \beta\sigma_{12} \geq \tau_{23} + \beta\sigma_{23} \quad (2.2a)$$

$$F' = \tau_{13} + b\tau_{23} + \beta(\sigma_{13} + b\sigma_{23}) = C \quad \text{when} \quad \tau_{12} + \beta\sigma_{12} \leq \tau_{23} + \beta\sigma_{23} \quad (2.2b)$$

where τ_{12} , τ_{23} and τ_{13} are the principal shear stresses, $\tau_{13} = (\sigma_1 - \sigma_3)/2$, $\tau_{12} = (\sigma_1 - \sigma_2)/2$ and $\tau_{23} = (\sigma_2 - \sigma_3)/2$; σ_{12} , σ_{23} and σ_{13} are the corresponding normal stresses on the principal shear stress element; σ_1 , σ_2 and σ_3 are the principal stresses, $\sigma_1 \geq \sigma_2 \geq \sigma_3$, b is a weighted coefficient, reflecting the relative effect of the intermediate principal shear stress τ_{12} or τ_{23} on the strength of materials; C equals to the material strength. Denoting the tension-compression strength ratio as $\alpha = \sigma_t/\sigma_c$, we rewrite Eq. (2.2a) and (2.2b) in terms of principal stresses as follows

$$F = \sigma_1 - \frac{\alpha}{1+b}(b\sigma_2 + \sigma_3) = \sigma_t \quad \text{when} \quad \sigma_2 \leq \frac{\sigma_1 + \alpha\sigma_3}{1+\alpha} \quad (2.3a)$$

$$F' = \frac{1}{1+b}(\sigma_1 + b\sigma_2) - \alpha\sigma_3 = \sigma_t \quad \text{when} \quad \sigma_2 \geq \frac{\sigma_1 + \alpha\sigma_3}{1+\alpha} \quad (2.3b)$$

2.2.3 Elastoplastic Limit Analysis

2.2.3.1 Outer Steel Tube

The axial performance test only focused on the CFDST column with inner circular-outer square section (Type *b*). In order to derive an equation based on the limit equilibrium method, a regression analysis considering steel ratio and confinement factor can be obtained by the test data. But confinement mechanism of square steel tube is more complex than circular tube, so the square tube needs to be equivalent to the circular steel tube in the calculation, and an equivalent confining reduction factor ζ and a concrete strength reduction factor γ_μ are considered to compensate this transition.

By the principle of same area, the cross-section of square steel tube can be transformed into the circular steel tube. The formulas are shown as follows

$$D_o = 2B/\sqrt{\pi} = 1.1284B \quad t_o = r_o - (B - 2t_s)/\sqrt{\pi} = r_o - 0.5642(B - 2t_s) \quad (2.4)$$

where B , t_s are the side length and thickness of the square steel tube, and D_o , t_o are diameter and thickness of the equivalent outer steel tube respectively. Then, the stress model of inner circular-outer circular section (Type *a*) is shown in Fig. 2.4 when the composite column is under compression, where p means the confining pressure. It can be seen that outer concrete is confined only by the outer tube, while inner concrete is confined by both tubes. As for the inner circular-outer square section

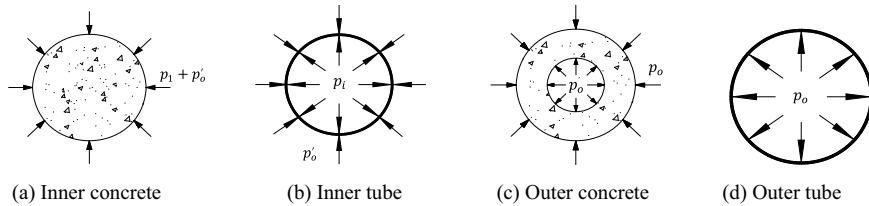


Fig. 2.4 Simplified mechanical models of the CFDST column

(Type *b*), the equivalent outer concrete and outer steel tube will be considered after the transformation of the square tube to a circular one.

2.2.3.2 Concrete

The inner concrete was filled in a circular tube, its stresses can be explicated by $0 > \sigma_1 = \sigma_2 > \sigma_3$, $\sigma_1 = \sigma_2 = p_i + p_o'$. When $\sigma_2 \geq \frac{\sigma_1 + \alpha \sigma_3}{1 + \alpha}$, substitute them into the stress expression of TUST, the following expression can be obtained

$$\sigma_3 = f_c + k_c (p_i + p_o') \quad (2.5)$$

where f_c is the standard strength of concrete; k_c as a strength improvement coefficient of concrete under fixed lateral compressive force has been studied much. In TUST, k_c can be calculated by cohesion and the angle of friction at a material failure state. According to the test of Richart et al. (1928), k_c has been taken as the constant 4.1 simply here.

The outer concrete can be assumed as a thick-walled cylinder with an inner diameter D_i and an outer diameter D_o , which is subjected to internal pressure p_o' and external pressure p_o . The elastic stress distribution (Lame's Equations) has the form of

$$\sigma_1 = \frac{D_i^2}{D_o^2 - D_i^2} \left(1 + \frac{D_o^2}{4r^2} \right) p_o' - \frac{D_o^2}{D_o^2 - D_i^2} \left(1 + \frac{D_i^2}{4r^2} \right) p_o \quad (2.6a)$$

$$\sigma_2 = \frac{D_i^2}{D_o^2 - D_i^2} \left(1 - \frac{D_o^2}{4r^2} \right) p_o' - \frac{D_o^2}{D_o^2 - D_i^2} \left(1 - \frac{D_i^2}{4r^2} \right) p_o \quad (2.6b)$$

The relationship between p_o' and p_o was deduced as $p_o' = p_o \frac{D_o}{D_i}$, Eqs. (2.6a) and (2.6b) can be changed to $\sigma_1 = \sigma_2 = p_o$ when $p_o' = p_o$. Because p_o' is larger than p_o , the assumption is reasonable and safe for applications. Therefore, the ultimate strength of outer concrete can be expressed by

$$\sigma_3 = f_c + k_c p_o \quad (2.7)$$

Therefore, the ultimate strength of concrete has the same expression (see Eq. 2.5). The difference is the value of confining pressure, showing that outer concrete is confined by the outer steel tube while inner concrete is confined by both inner and outer steel tubes.

The confinement of square steel tube to concrete is very uneven along its side (Han and Yang 2003), strong at the four corners and weak along the middle edge. Therefore, the equivalent reduction factor ζ should be considered to reduce the confinement of the equivalent circular steel tube (Li and Zhao 2006). Denoting the thickness-side ratio $\nu = t/B$, the expression of the equivalent reduction factor is $\zeta = 66.4741\nu^2 + 0.9919\nu + 0.41618$. Actually, the confining pressure to the infilled concrete in the square tube is

$$\bar{p}_o = \zeta p_o \quad (2.8)$$

As for the inner concrete, the confining pressure can be written as $\bar{p}_i = p_i + \bar{p}'_o = p_i + \bar{p}_o \frac{D_o}{D_i} = p_i + \zeta p_o \frac{D_o}{D_i}$, so the actual compressive strength of inner concrete can be expressed by

$$\bar{f}_{ci} = f_c + k_c \left(p_i + \zeta p_o \frac{D_o}{D_i} \right) \quad (2.9)$$

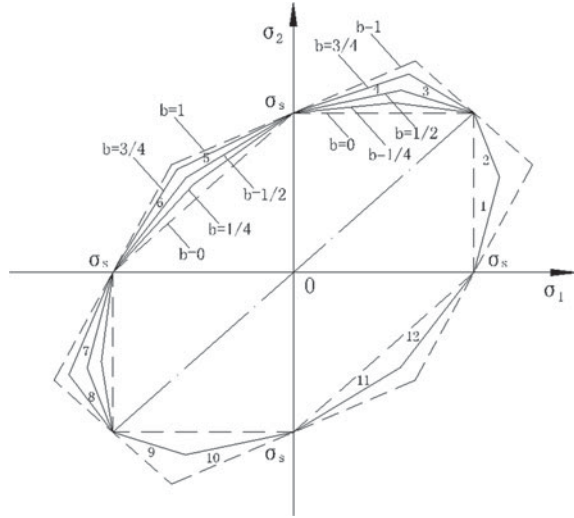
Meanwhile, there are effective and non-effective confining zones of concrete inside the square steel tube. In this book, concrete strength reduction factor is taken as $\gamma_\mu = 1.67 D_o^{-0.112}$ in order to homogenize these two effects. In the above formula, D_o is the outside diameter of equivalent circular steel tube. Therefore, the actual compressive strength of outer concrete can be expressed by

$$\bar{f}_{co} = \gamma_\mu \sigma_3 = \gamma_\mu (f_c + k_c \zeta p_o) \quad (2.10)$$

In the Eqs. (2.9) and (2.10), the unknown parameter is only the confining pressure, it can be deduced by the analysis of steel tube subsequently. Under the state of limit equilibrium, both outer and inner steel tubes are in three-dimensional stress states including axial compression, radial compression and circumferential tension. In most CFST structures, the diameter-thickness ratio or side length-thickness ratio of steel tubes is generally larger than 20, and it can be regarded as a thin-walled cylinder. Therefore, the steel tube can be calculated under a plane stress state, i.e. radial compressive stress $\sigma_r = 0$.

According to TUST, the stresses of steel tube in a plane state are shown in Fig. 2.5. When b varies from 0 to 1, a series of convex yield criteria suitable for any kind of materials are deduced. In particular, the TUST becomes the Tresca criterion when $\alpha = 1.0$ and $b = 0$. The von Mises criterion can be linearly approximated to the TUST with $\alpha = 1.0$ and $b = 0.366$, and the Mohr-Coulomb criterion is obtained with $b = 0$ (Yu 2004). No matter what b takes for different strength theories, if circumferential tensile stress σ_θ reaches the yield strength, axial compressive stress σ_z nearly becomes zero, and vice versa.

Fig. 2.5 Yield criterion in a plane state



According to the test investigation, both the outer tube and inner tube cannot reach this ideal state under ultimate state of the CFDST column. The steel tube can still bear some vertical load under the ultimate state of the whole column, and assume β_f as the strength reduction factor of steel tube, that is

$$\sigma_3 = \sigma_z = \beta_f f_y \quad (2.11)$$

Therefore, the stress state of steel tubes can be explicated by $\sigma_3 = \sigma_z = -\beta_f \sigma_s$, $\sigma_2 = \sigma_r = 0$, $\sigma_1 = \sigma_\theta$. For $|\sigma_3| > \sigma_1$ and $\sigma_2 \geq \frac{\sigma_1 + \sigma_3}{1 + \alpha}$, substitute them into the stress expression of TUST (see Eqs. 2.3a, b), the following expression can be obtained.

$$\frac{\sigma_\theta}{1 + b} + \sigma_z = f_y \quad (2.12)$$

Meanwhile, the confining pressure on the concrete coming from the circular steel tube can be written as

$$p = \frac{2t}{D} \sigma_\theta \quad (2.13)$$

The strain value of steel tube has been obtained at the ultimate state through the specimens' test. Then the circumferential tensile stress and vertical stress of the steel tube can be derived by the plastic theory and TUST. Therefore, the following expression can be derived as

$$\beta_f = \frac{(u + 2)(1 + b)}{3(1 + u) + b(u + 2)} \quad (2.14)$$

Table 2.2 Calculation of vertical stress of steel tube under the ultimate state

Specimen	T	D	σ_s	ε_z	ε_θ	μ	$b = 0$		$b = 0.366$		$b = 1$	
							σ_z	β_f	σ_z	β_f	σ_z	β_f
G1-2	1.4	58.5	352.5	2917	1232	0.43	200.1	0.57	226.2	0.64	255.3	0.72
G1-2	1.4	58.5	352.5	3375	1606	0.48	197.1	0.56	223.4	0.63	252.8	0.72
G1-2	1.4	58.5	352.5	3214	1477	0.46	198.0	0.56	224.2	0.64	253.6	0.72
G1-2	1.4	58.5	352.5	3767	1983	0.53	194.5	0.55	220.9	0.63	250.7	0.71
G1-3	0.9	74	680	2818	1218	0.43	384.9	0.57	435.3	0.64	491.6	0.72
G1-3	0.9	74	680	3382	1475	0.44	384.5	0.57	435.0	0.64	491.2	0.72
G1-3	0.9	74	680	3165	1172	0.37	392.1	0.58	442.0	0.65	497.4	0.73
G1-3	0.9	74	680	3655	1397	0.38	390.7	0.57	440.7	0.65	496.2	0.73
G1-4	0.9	83	597	3127	823	0.26	356.5	0.60	399.5	0.67	446.4	0.75
G1-4	0.9	83	597	3495	1020	0.29	353.0	0.59	396.3	0.67	443.7	0.74
G1-4	0.9	83	597	3910	1206	0.31	351.1	0.59	394.4	0.66	442.2	0.74
G1-4	0.9	83	597	3799	934	0.25	358.7	0.60	401.5	0.67	448.2	0.75

where $u = \varepsilon_z / \varepsilon_\theta$. $\varepsilon_z, \varepsilon_\theta$ are the vertical and circumferential strain of the steel tubes. The strain obtained from different gauging points in the experiment is shown in Table 2.2 when b takes different values. For steel, we can take the approximation of von Mises criterion as the final answer. In this book, β_f was taken as 0.65 in the calculation of bearing capacity of the steel tubes. Then synthesize Eqs. (2.12) and (2.13), σ_θ and confining pressure p can be deduced as follows

$$\sigma_\theta = 0.4774 f_y \quad p = 0.9548 \frac{t}{D} f_y \quad (2.15)$$

2.2.3.3 Inner steel

As can be seen in Figs. 1.7–1.10, when the composite column is under axial compression, the inner steel including section steels and steel tubes can be considered under three-dimensional compression, as shown in Fig. 2.6.

Then the stress state of the inner steel is: $0 > \sigma_1 = \sigma_2 = -p > \sigma_3, \sigma_3 = \sigma_z$. For steel, $\alpha = 1$, so $\sigma_2 \geq \frac{\sigma_1 + \alpha \sigma_3}{1 + \alpha}$, and the axial compressive strength of steel under three-dimensional compression can be calculated as bellow according the expression of TUST (Eq. 2.3a, b).

$$\sigma_3 = \sigma_z = -f_y - p \quad (2.16)$$

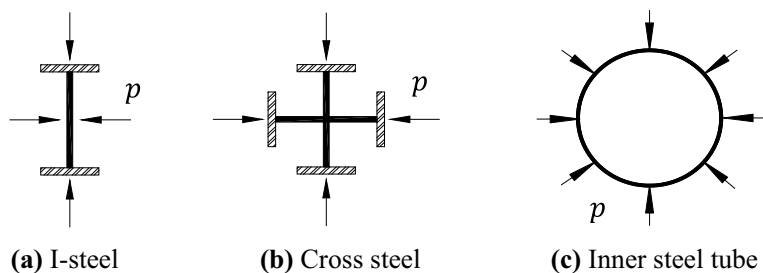


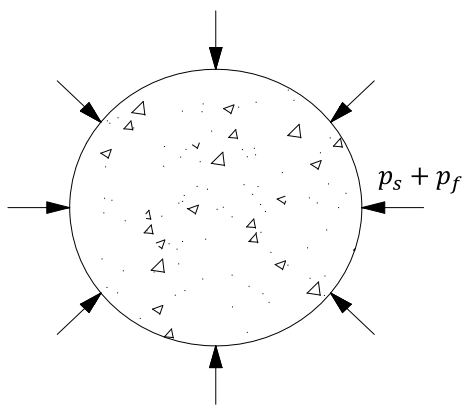
Fig. 2.6 Three-dimensional compression of steel

Moreover, most of the steel is thin-walled, the value of p is always smaller than 10 % of steel yield strength (f_y). Therefore, the yield strength of inner steel is taken as the axial strength simply.

2.2.3.4 CFRP sheet

For the CFRP-confined CFST column, the compressive mechanism and physical properties were analyzed aiming at investigating the confinement effect of CFRP on CFST columns (Guo and Zhang 2018). Although the CFRP cylinder has no direct contribution to the axial bearing capacity, the transverse fiber sheets contribute to the strength enhancement by confining the CFST column in whole (see Types *g*, *i* in Fig. 1.9) or in part concrete (see Type *h* in Fig. 1.9), leading to a higher compressive strength of the column. As we all know, during the process of compression of the composite columns, there exists horizontal deformation when vertical load acts on the whole section. Take the example of Type *g* in Fig. 1.9, concrete is filled in the circular tube wrapped by the CFRP sheet, so its simplified model of stress for concrete can be plotted in Fig. 2.7. Using the above method, the ultimate strength of concrete

Fig. 2.7 Stress model of concrete in the CFRP-confined CFST column



in the CFRP-confined CFST column is changed as

$$\sigma_3 = f_c + k_c(p_s + p_f) \quad (2.17)$$

where p_s, p_f are confining pressure from the steel tube and CFRP respectively, $p_f = \frac{2t_{cf}}{D_{cf}} f_{cf}$; f_{cf} is tensile strength of CFRP; t_{cf} is the wall thickness of CFRP cylinder; D_{cf} is the diameter of CFRP cylinder.

Finally, combining the above calculation process, we can obtain the basic form of the ultimate bearing capacity including four parts. Square steel tubes are used in several cross-sections of reinforced CFSTs in Figs. 1.7–1.10, so after the transformation from square to circular (Eq. 2.4) and consideration of confinement of the CFRP sheet, Eq. (2.1) may have several expression forms. As for CFDST, the ultimate axial bearing capacity obtained by the elastoplastic limit equilibrium method is

$$N_C^1 = f_{si} A_{si} + \beta_f f_{so} A_{so} + \left[f_c + k_c \left(p_i + \zeta p_o \frac{D_o}{D_i} \right) \right] A_{ci} + \gamma_\mu (f_c + k_c \zeta p_o) A_{co} \quad (2.18)$$

2.3 Calculation by Unified Theory of CFST

2.3.1 Method of Unified Theory of CFST

The Unified Theory of CFST was presented by Prof. Zhong Shan-tong in 2006 (Zhong 2006). CFST is considered as a unified body, a new composite material used to study its mechanical behaviors. It is a new method to simplify the design work. In the past years, the method was applied to determine the behavior of common CFST members under different loadings. The method has given a unified formula for various members by means of regression analyses based on the tests considering steel ratio and confinement factor. It defined the confinement coefficient ξ as the confinement effect to concrete coming from a circular steel tube. The expression for common circular CFST column is $\xi = A_s f_s / (A_c f_{ck})$, in which, A_s and A_c are the cross section area of the circular steel tube and concrete; f_s and f_{ck} are the strength of steel and concrete respectively.

Based on this idea and its engineering models, the load-bearing capacity of the common CFST can be predicted. In common CFST structures, confined concrete shows greatly increased maximum compressive capacity, increased stiffness and extended strain at the peak stress. And this working mechanism is also suitable for the composite CFST because the concrete area is still predominant in the overall cross section. The main cause of using steels or CFRP is that the composite column can utilize the advantages of both confining material and concrete; viz. steel or

CFRP members have high tensile strength and ductility, while concrete members are advantageous in compressive strength and stiffness. Therefore, the improvement of ultimate bearing capacity of the reinforced CFST columns depends largely on the confinement to the concrete. Basically, in all reinforced CFST structures, effective confined concrete can change its failure mode from a brittle one to a relatively ductile one. Meanwhile, the reaction of this confinement can make local buckling of the steel tubes either delayed or restrained. And as discussed by Tao et al. (2005), the CFRP cylinder can also impede buckling of the stub column, leading to dramatic improvements in buckling and post buckling behavior of the entire system. Therefore, because the concrete is still confined by the outermost steel tube, and this confining effect is more or less strengthened by other confining members, the methodology of Unified Theory can also be applied to reinforced CFST members. One composite material can also be considered to assess its behaviors, but there are multi-steel or multi-material to confine concrete in the composite CFST structure, so the confining effect deriving from the steel tube and other confining material should be reevaluated.

2.3.2 Equivalent Confinement Coefficient

One equivalent confinement coefficient is proposed to evaluate the confinement between concrete and confining material, reflecting the physical composition and geometric properties of the composite columns. The confining effect has a great relationship with the shape of the confining material (Zhang and Zhang 2015, 2016), for example, confining force is distributed evenly for the circular section; while for the square section, confining force is the least around the middle of the straight sides because the straight sides can be easily bended and confining force is larger at the corners; it is the same for the octagon section that confining force is large at the corners and small around the middle of the sides. As a result, when the amount of confining material is not changed, the confining degree from different-shaped sections is different. The circular section has the largest value, the quadrangle has the least one and the polygon has the middle one. For I-shaped section steel, the confining effect can be considered as three sides of the equivalent square. Similarly, for cross-steel, the confining effect can be considered equivalent to the corresponding square section.

The confining degree with different cross sections can be reflected by the confinement coefficient. We take the equivalent confinement coefficient for every cross-section in the calculation of strength, and the confining effect can be reduced by being multiplied a reduction factor of cross section (k_s) for non-circular cross sections. Because there are multi-materials to confine concrete in the CFST structure, the confining effect is strengthened inevitably, and the equivalent confinement coefficient can be obtained by combining the confinements from every confining material. The expression is

Table 2.3 Values of k_s

Section types	Steel Q235 or Q345					Steel Q390				
	0.05	0.10	α	0.15	0.20	0.05	0.10	α	0.15	0.20
Circular			1					1		
Octagonal			0.8			0.89	0.90		0.92	0.94
Square	0.74	0.73		0.73	0.72	0.74	0.73		0.72	0.71
I-shaped	0.52	0.51		0.51	0.50	0.52	0.51		0.51	0.50

$$\xi_{ssc} = \frac{\sum k_s A_m f_m}{A_c f_{ck}} \quad (2.19)$$

where f_m is yield strength of confining material; A_m is cross section area of confining material; k_s is reduction factor of cross section as shown in Table 2.3 (Chen 2001; Zhong 1996), and α is the steel ratio of the composite structure which can be written as $\alpha = \sum A_s / A_c$. And for the CFRP cylinder, $k_s = 1$. As can be seen in the Eq. (2.19) that the confinements to concrete are from all the confining materials, so the physical composition and geometric properties are considered into the equivalent confinement coefficient. The equivalent confinement coefficient can not only reflect confining material ratio but also mainly reflect the confinement to the concrete in reinforced CFST.

2.3.3 Calculation Formulas

For various sections of reinforced CFSTs, composite performance and design specifications are comprehensively reflected by the equivalent confinement coefficient, so a unified formula of compressive strength for CFDST can be used without consideration of the combination of different cross sections. Then the compressive strength (f_{ssc}) of reinforced CFST columns can be calculated by

$$f_{ssc} = (1.212 + a\xi_{ssc} + b\xi_{ssc}^2) f_{ck} \quad (2.20)$$

where a and b reflect the contributions of confining material and concrete respectively. They can be calculated by

$$a = 0.1759 f_{ss} / 235 + 0.974 \quad (2.21)$$

$$b = -0.1038 f_{ck} / 20 + 0.0309 \quad (2.22)$$

where, f_{ss} is the weighted average yield strength of the confining material, which is written as

$$f_{ss} = \frac{\sum A_m f_m}{\sum A_m} \quad (2.23)$$

Therefore, the ultimate bearing capacity of the CFDST column subjected to axial compression can be calculated as following

$$N_C^2 = f_{ssc} A_{ssc} \quad (2.24)$$

where A_{ssc} is the section area of the CFDST column.

2.4 Validation and Analysis

A lot of stub columns of four types of reinforced CFST as shown in Figs. 1.7–1.10, were tested by many researchers, and the corresponding calculation results of bearing capacity using the method of Unified Theory and limit equilibrium theory are listed in Table 2.4, where f is defined as strength of material, A is defined as the area of cross-section and t means wall thickness; the subscripts s , c , o , i and cf in the table are the abbreviations of steel, concrete, outer, inner and CFRP respectively. The calculation results (N_C^1 , N_C^2) derived from Eqs. (2.18) and (2.24) are compared with the experimental values (N_t). The axial compressive bearing capacity obtained by the simple superposition method is written as N_0 , it means the interaction between concrete and the steel is not considered.

$$N_0 = \sum f_s A_s + f_c A_c \quad (2.25)$$

According to the cross-section types a – m for reinforced CFSTs I–IV given in Sect. 1.5, nearly one hundred test specimens were investigated from the references. The comparison between these calculated results and corresponding test results is shown in Fig. 2.8. It can be found that all the points are basically distributed in the vicinity of the angle bisector ($y = x$), and that means the calculated value is very close to the experimental value. Thus, these two methods can both be applied to investigate the axial bearing capacity of reinforced CFST stub columns. Meanwhile, most of the calculated values are slightly lower than experimental results, so the method has been verified feasible and conservative to investigate the ultimate bearing capacity of CFDST columns. The mean of N_C^1/N_t is 0.983, and mean squared error is 0.065; the mean of N_C^2/N_t is 0.966, and mean squared error is 0.103. This indicates that N_C^1 obtained by the limit equilibrium method is more accurate and reliable than N_C^2 obtained by the method of the Unified Theory of CFST. But the method of the Unified Theory of CFST is simple and easy to realize since it just considers the reinforced

Table 2.4 Test parameters and calculated results

Part A: Reinforced CFSTs I, II, IV. Types a–f, j–m in Figs. 1.7, 1.8, and 1.10												
Type	Number	$A_{so}(\text{mm}^2)$	$f_{so}(\text{MPa})$	$A_{st}(\text{mm})^2$	$f_{st}(\text{MPa})$	$A_c(\text{mm})^2$	$f_c(\text{MPa})$	$N_f(\text{kN})$	$N_0(\text{kN})$	$N_f^1(\text{kN})$	$N_f^2(\text{kN})$	References
a	DSC2-1	2827.4	478	1885	478		44.42	8300	6387.5	7791.4	7771.5	Zhang et al. (2004)
	DSC1	7045	234	2035.7	234	65,160	26.8	5179	4373.7	4675.2	5302.7	Jiang (2007)
	DSC2	7045	234	2387.6	234	64,808	26.8	5538	4475.4	4773.0	5492.8	
	DSC3	7045	234	3392.9	234	63,803	26.8	5923	4766.1	5050.8	5752.7	
	DSC4	7045	234	7351.4	234	59,844	26.8	7128	5910.8	6102.2	7802.7	
b	DSC5	7045	234	6192.2	234	61,004	26.8	7590	5575.6	5803.2	7411.2	
	G1-2	1248	407.5	255.1	352.5	12,897	23.6	980	917.6	964.9	1031.0	Zhang et al. (2013)
	G1-3	1248	407.5	209.2	680.0	12,943	23.6	1040	899.2	989.4	1111.9	
	G1-4	1248	407.5	234.7	597.0	12,917	23.6	1080	908.7	1000.6	1113.0	
	III-CSCFT1	2606.4	348	727.0	314	9119	62.4	3198	2952.3	3535.1	3332.4	Qian et al. (2011)
c	III-CSCFT2	2606.4	348	1199.8	328	28,646	62.4	3415	3088.1	3721.1	3552.2	
	III-CSCFT3	2606.4	348	1746.1	308	28,099	62.4	4130	3198.3	3872.9	3792.1	
	III-CSCFT4	3888	338	727.0	314	27,902	62.4	4021	3283.5	3875.5	3800.7	
	III-CSCFT5	3888	338	1200.0	328	27,429	62.4	4165	3419.2	4050.1	3906.3	
	III-CSCFT6	3888	338	1746.1	308	26,883	62.4	4436	3529.4	4191.0	3999.2	
	III-CSCFT7	3888	338	928.3	324	27,700	62.4	3900	3343.4	3953.3	3879.8	
	GG-2	6861.2	307	6421.4	310.7	48,733	27.1	7840	5497.0	6549.5	7867.5	Cai (2003)
	GG-1*	6861.2	307	2607.5	274.4	48,148	27.1	6360	4591.5	5672.9	5776.7	
				917.3	310							(continued)

Table 2.4 (continued)

Part A: Reinforced CFSTs I, II, IV. Types $a-f, j-m$ in Figs. 1.7, 1.8, and 1.10												
Type	Number	$A_{so}(\text{mm}^2)$	$f_{so}(\text{MPa})$	$A_{st}(\text{mm})^2$	$f_{st}(\text{MPa})$	$A_c(\text{mm})^2$	$f_c(\text{MPa})$	$N_t(\text{kN})$	$N_0(\text{kN})$	$N_y^1(\text{kN})$	$N_y^2(\text{kN})$	References
d	NS-A	1408	318	2324	288	18,620	29.6	2100	1668.2	1920.4	1961.6	Zhao (2003)
	HS-A	1408	318	2324	288	18,620	47.9	2650	2009.0	2393.3	2485.5	
	HS-B	1953	318	2324	288	18,877	47.9	2835	2194.5	2619.0	2717.9	
	HS-C	2036	269	2324	288	33,309	47.9	3550	2812.4	3414.2	3354.4	
e	S5L10I	4169	288	1433	338	32,423	48.4	3620	3254.3	3610.2	3454.3	Wang et al. (2005)
	S4L10I	3429	289	1433	338	33,163	48.4	3410	3080.4	3424.3	3181.6	
f	S5L10V	4169	288	2866	338	30,990	48.4	4035	3669.3	4104.2	3712.3	
	S5H10V	4169	288	2866	338	30,990	70.8	4880	4363.4	5116.9	4370.6	
	S4L10	3429	289	2866	338	31,730	48.4	3930	3495.4	3737.0	3695.6	
	S4H10	3429	289	2866	338	31,730	70.8	4750	4206.2	4946.9	4417.4	
	S4H14	3429	289	3870	327	30,726	70.8	4710	4431.9	5176.3	4339.2	
	S5L10C	4169	288	2866	338	30,990	48.4	3860	3669.3	4104.2	3779.3	
j	S5H10C	4169	288	2866	338	30,990	70.8	4980	4363.5	5116.9	4875.2	Wei et al. (1995)
	D5-1	205	409	105	432	4652	45.6	443	341.3	375.1	375.1	
	E4-1	580	262	223	216	6501	45.6	598	496.6	554.4	554.4	
	Cc2	1696.5	275.9	452.4	396.1	23,637	36.0	1790	1398	1578.3	1856.8	
	Cc3	1696.5	275.9	829.4	370.2	19,365	36.0	1649	1390.1	1369.6	1717.6	Tao et al. (2004)
	Cc4	1696.5	275.9	1319.5	342	10,053	36.0	1396.5	1281.5	1331.4	1279.7	

(continued)

Table 2.4 (continued)

Part A: Reinforced CFSTs I, II, IV. Types a–f, j–m in Figs. 1.7, 1.8, and 1.10												
Type	Number	$A_{so}(\text{mm}^2)$	$f_{so}(\text{MPa})$	$A_{st}(\text{mm})^2$	$f_{st}(\text{MPa})$	$A_c(\text{mm})^2$	$f_c(\text{MPa})$	$N_r(\text{kN})$	$N_0(\text{kN})$	$N_y^1(\text{kN})$	$N_y^2(\text{kN})$	References
<i>k</i>	Cc5	1074.4	275.9	546.6	374.5	7565	36.0	901	793.7	863.1	890.2	
	Cc6	2261.9	275.9	1074.4	294.5	35.031	36.0	2440	2202.5	2610.3	2624.7	
	SC	6400	319.83	626.7	344.98	34.312	24.9	3174	3117.3	3520	3098.9	Wang and Cheng (2005)
	SCC1*	4160	345.12	2297.8	382.09	32.014	25.4	3557	3342.7	3667.2	3777.4	
<i>l</i>	Asc2	1440	275.9	301.6	415.42	12.885	31.4	1057	927	1029.2	947.9	Huang and Zhang (2007)
	Asc3	1440	275.9	547	369.6	11.047	31.4	995	946	971.5	998.7	
	Asc5	2160	275.9	829	376.72	25.247	31.4	1717	1670	1787.0	1698.7	
	Asc6	2880	275.9	1074	312.3	45.962	31.4	2520	2521	2779.0	2777.1	
	CS1	2382	485	488	477	6293	55.8	1580	1739	1484.3	1409.5	Zhao and Grzebieta (2002)
	CS2	1612	445	488	477	6808	55.8	1202	1330	1367.9	1184.4	
<i>m</i>	CS3	1178	464	488	477	6956	55.8	1044	1167.5	1253.3	1088.9	
	CS4	828	453	488	477	7183	55.8	830	1008	1121.2	986.3	
	1	2016	300	810	325	23.203	37	1790	1726	2020.4	1859.0	Yang et al. (2004)
	2	2132	300	989	260	24.849	37	1904	1816	2110.2	1911.7	
	3	2347	300	989	260	33.703	37	2580	2208	2586.1	2365	
	4	2576	300	1046	365	42.302	37	3000	2720	3215.7	2896.2	

(continued)

Table 2.4 (continued)

Part B: Reinforced CFST III. Types $g-i$ in Fig. 1.9

Type	Number	t_{cf} (mm)	f_{cf} (MPa)	t_s (mm)	A_s (mm) ²	f_s (MPa)	f_{ck} (MPa)	N_r (kN)	N_0 (kN)	N_y^I (kN)	N_y^J (kN)	References
g	1-1.5	0.17	1260	1.5	598.5	350	40.15	1086	706.1	993.0	1096.9	Gu and Li (2011)
	1-2.5	0.17	1260	2.5	1013.2	350	40.15	1294	859.2	1164.8	1293.7	
	1-3.5	0.17	1260	3.5	1440.4	310	40.15	1348	959.1	1176.5	1408.0	
	1-4.5	0.17	1260	4.5	1880.2	310	40.15	1698	1103.6	1446.2	1575.6	
	2-1.5	0.34	1260	1.5	598.5	350	40.15	1283	706.1	1135.0	1243.5	
	2-2.5	0.34	1260	2.5	1013.2	350	40.15	1506	859.2	1293.3	1430.9	
	2-3.5	0.34	1260	3.5	1440.4	310	40.15	1593	959.1	1395.0	1540.1	
	2-4.5	0.34	1260	4.5	1880.2	310	40.15	1846	1103.6	1505.4	1702.0	
h	SC41	0.167	1500	4	2400	295	53.6	2215	1850.5	2175.8	2341.1	Li et al. (2008)
	SC42	0.334	1500	4	2400	295	53.6	2275	1850.5	2261.3	2443.7	
	SC51	0.167	1500	5	3000	295	53.6	2485	2011.9	2326.4	2477.8	
	SC52	0.334	1500	5	3000	295	53.6	2585	2011.9	2407.9	2356.7	
	SC61	0.167	1500	6	3600	295	53.6	2710	2173.4	2472.8	2801.1	
	SC62	0.334	1500	6	3600	295	53.6	2775	2173.4	2550.0	2677.3	
i	A-1	0.111	690	3.5	1960	300	22.3	1107	982.5	1166.0	1110.7	Wang and Shao (2014)
	A-2	0.222	690	3.5	1960	300	22.3	1129	982.5	1272.3	1192.6	
	A-3	0.333	690	3.5	1960	300	22.3	1222	982.5	1380.2	1285.4	

(continued)

Table 2.4 (continued)

Part B: Reinforced CFST III. Types g-i in Fig. 1.9

Type	Number	t_{cf} (mm)	f_{cf} (MPa)	t_s (mm)	A_s (mm) ²	f_s (MPa)	f_{ck} (MPa)	N_f (kN)	N_0 (kN)	N_y^1 (kN)	N_y^2 (kN)	References
	B-1	0.111	690	3.5	1960	300	26.4	1200	1055.0	1260.5	1228.5	
	B-2	0.222	690	3.5	1960	300	26.4	1237	1055.0	1365.7	1266.3	
	B-3	0.333	690	3.5	1960	300	26.4	1294	1055.0	1472.6	1305.6	
	C-1	0.111	690	3.5	1960	300	32.8	1204	1168.2	1409.3	1297.1	
	C-2	0.222	690	3.5	1960	300	32.8	1300	1168.2	1513.8	1352.5	
	C-3	0.333	690	3.5	1960	300	32.8	1400	1168.2	1619.8	1405.9	
	D-1	0.111	690	3.5	1960	300	40	1601	1295.6	1578.0	1502.1	
	D-2	0.222	690	3.5	1960	300	40	1742	1295.6	1682.2	1655.4	
	D-3	0.333	690	3.5	1960	300	40	1815	1295.6	1787.8	1797.6	

Note Specimen with * represents double tubes inside

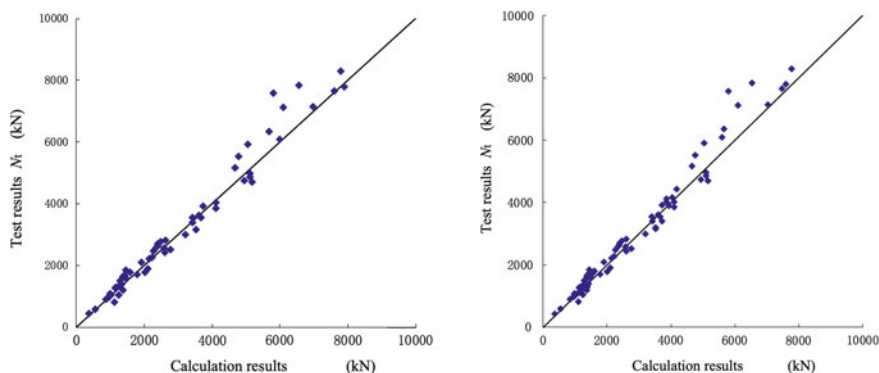


Fig. 2.8 Calculated results versus experimental results

CFST column as one composite material, while the method of limit equilibrium method sounds complicated since it applies TUST to analyze every component of the composite column.

By comparing the test results with the results (N_0) obtained by the simple superposition method, a bearing capacity enhancement rate is defined as the expression as follow. The confining effect from the confining materials is not considered in the simple superposition method, so N_0 is just the sum of bearing capacity of the steel and concrete. While for CFRP sheets, it has no contribution in the calculation N_0 . As can be seen in the Table 2.4, the mean of N_0/N_t for every type of reinforced CFST is very different between each other, and the value of Form I, II, III, IV is respectively 0.81, 0.87, 0.76, 0.95, and mean squared error is 0.84. This means there is a wide variation in estimates by the simple superposition method except for hollow CFDST. The interaction between steel and concrete in hollow CFDST is the least because the sandwiched concrete cannot obtain effective confinement from the double steel tubes. Because the effect of CFRP on axial compressive strength is not reflected in the simple superposition method, N_0/N_t gets the minimum. Because the CFRP sheet is very thin, it is demonstrated that the bearing capacity of composite columns improves much than the corresponding pure CFST columns with nearly the same cross-section area. It sounds the most effective to use CFRP to strengthen CFST. However, in consideration with the construction convenience and cost, using an inner steel tube to reinforce CFST is more applicable than CFRP.

2.5 Parametric Study for CFDST Column

CFDST was testified to be the more applicable CFST structure by comparison of mechanical properties and steel ratio among all the various sections of reinforced CFST columns. The CFDST column has the advantages of larger bearing capacity, greater plasticity, upstanding toughness and good fire resistance. In particular, the

section of inner circular-outer square (Type b) is considered as more ideal one, because it takes the advantage of both good confinement of circular steel tubes to concrete and the convenient connection construction between square steel tubes and frame beams. According to some references (Zhang et al. 2013; Pei 2005), these excellent structural and fire-resistant properties make them particularly suitable to apply in high-rise buildings. Furthermore, this column, without needing to consider the limit of axial compression ratio, can reduce the column size to provide substantial benefits where floor space is at a premium such as in car parks and office blocks. Therefore, the design provisions and structural contributions of the circular steel tube need to be analyzed as below.

2.5.1 Influence of Inner Tube

In order to further study the reinforcing function of inner circular steel tube inside of CFDST, this book has changed the diameter-thickness ratio of inner tube with no change of the steel ratio and steel strength of the composite column, so different axial bearing capacity N can be obtained for CFDST with different diameter-thickness ratios of inner tube.

Take the test specimen in Sect. 2.2, while N_0 is the bearing capacity of the steel tube and concrete without considering the interaction between steel tubes and concrete, as shown in Eq. (2.25). The relationship between N/N_0 and the parameter of inner steel tube is shown in Figs. 2.9 and 2.10, herein the calculation results (N) are obtained by the limit equilibrium method (N_C^1). N/N_0 is the enhanced coefficient of bearing capacity which reflects the function of interaction among outer, inner steel tubes and concrete. For every group, the steel ratio is the same, the value of N_0 is fixed, while

Fig. 2.9 Predicted axial load (N/N_0) versus D_i/B interaction curves

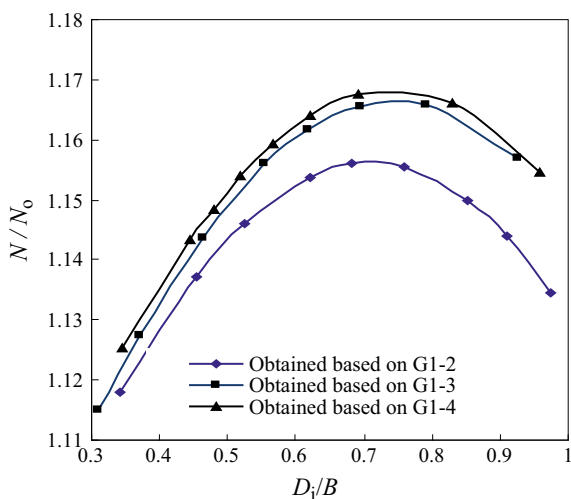
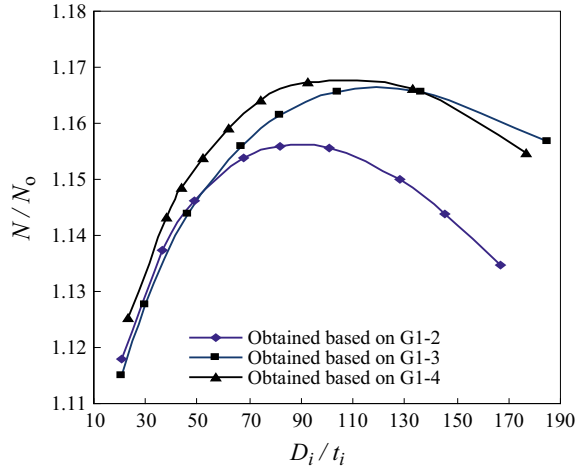


Fig. 2.10 Predicted axial load (N/N_0) versus D_i/t_i interaction curves



the calculated value N of the CFDST column has changed with different parameters. Therefore, when the value of N/N_0 is larger, it will be validated that the confinement mechanism of the column will be stronger and the bearing capacity will be improved more.

As can be seen from the curves, there are also optimal parameters that can make the inner steel tube play strongest confinement for the CFDST column. It can be seen that the optimal D_i/B to make the column with the maximum bearing capacity is near to 0.75. It demonstrates that the CFDST columns have the maximum capacities to resist axial compression when the ratio of D_i/t_i is between 80 and 100.

2.5.2 A Simple Model Used in Optimum Design

Because the more confinement concrete gets from tubes, the higher strength it has, we can analyze the deformation of concrete under the nearly ultimate state. There is volume expansion of the whole concrete.

$$\Delta V_c = \Delta V_{co} + \Delta V_{ci} \quad (2.26)$$

The minimum value of ΔV_c can be calculated since $\Delta V_c \geq 2\sqrt{\Delta V_{co}} \cdot \sqrt{\Delta V_{ci}}$:

$$\Delta V_{cmin} = 2\sqrt{\Delta V_{co}} \cdot \sqrt{\Delta V_{ci}} \quad \left(\text{only if } \sqrt{\Delta V_{co}} = \sqrt{\Delta V_{ci}} \right) \quad (2.27)$$

Hence, the CFDST column under axial compression has the highest load-capacity when $\Delta V_{co} = \Delta V_{ci}$.

A convenient and simple way to calculate the volume change of concrete is to assume $\Delta V_{co} = g(V_{co})V_{co}$ and $\Delta V_{ci} = g(V_{ci})V_{ci}$, where $g(\cdot)$ is a function of the concrete volume (Yuan et al. 2009). It can be seen that $g(\cdot)$ performs like an operator which is dependent on concrete volume. Therefore, according to Eq. (2.27), one obtains

$$g(V_{co})\left(B^2 - \frac{\pi D_i^2}{4}\right) = g(V_{ci})\frac{\pi D_i^2}{4} \quad (2.28)$$

Equation (2.28) can be rewritten as Eq. (2.29)

$$\frac{D_i}{B} = \sqrt{\frac{2}{\pi[1 + g(V_{ci})/g(V_{co})]}} \quad (2.29)$$

It must be mentioned that different forms for the function $g(\cdot)$ may be defined. Assuming $g(\cdot)$ is constant to define the simplest form, i.e. $g(V_{ci}) = g(V_{co})$, it can be obtained that $\frac{D_i}{B} = 0.798$. Therefore, the optimal D_i/B is very close to the result given in Fig. 2.10.

For the cross-section of inner circular-outer circular (Type a), the optimal design of the CFDST column can be obtained as follow. The condition as similar as Eq. (2.28) is changed as

$$g(V_{co})\frac{\pi}{4}(D_o^2 - D_i^2) = g(V_{ci})\frac{\pi}{4}D_i^2 \quad (2.30)$$

Then,

$$\frac{D_i}{D_o} = \sqrt{\frac{g(V_{co})}{g(V_{ci}) + g(V_{co})}} \quad (2.31)$$

Therefore, the optimal section design is $\frac{D_i}{D_o} = 0.707$ for the CFDST column with the inner circular-outer circular cross-section.

2.5.3 Contribution of Inner Tube

According to the above optimum design, the optimal circular steel tubes are supposed inside the pure square CFST columns, as shown in Table 2.5. The strength of the nominal inner tube is taken the same as the square steel tube. Then, the comparison of the ultimate bearing capacity was conducted between CFST columns and those reinforced by nominal inner tubes. By reviewing test results, the bearing capacity enhancement rate is described as the expression of $(N_C^1 - N_{CFST})/N_{CFST}$ as shown in Table 2.5. With the optimal circular steel tubes inside, the calculated results of

Table 2.5 Comparison of ultimate bearing capacity between CFST and CFDST columns

Specimens	Outer tubular $B \times t$	f_{co} (MPa)	Nominal Inner tubular $\Phi D_i \times t_i$	Inner tube D_i/t_i	f_c (MPa)	Test results N_{CFST} (kN)	Calculation results N_C^1 (kN)	$\frac{N_i - N_{CFST}}{N_{CFST}} \cdot 100\%$	References
CR4-C-8	215mm $\times$ 4.38mm	262	$\Phi 160\text{mm} \times 2.0\text{mm}$	80	53.8	3837	4810.6	25.4	Kenji et al. (2004)
CR4-D-4-2	323mm $\times$ 4.38mm	262	$\Phi 220\text{mm} \times 2.8\text{mm}$	79	27.5	4830	5770.9	19.5	
CR8-C-9	180mm $\times$ 6.6mm	824	$\Phi 120\text{mm} \times 1.5\text{mm}$	80	61.0	5870	6987.9	19.0	
sczs1-1-4	120mm $\times$ 3.84mm	330	$\Phi 80\text{mm} \times 1.0\text{mm}$	80	33.0	1080	1351.0	25.1	Han and Tao (2001)
sczs1-1-2	120mm $\times$ 3.84mm	330.1	$\Phi 80\text{mm} \times 1.0\text{mm}$	80	20.9	882	1094.8	24.1	
sczs1-2-4	140mm $\times$ 3.84mm	330.1	$\Phi 100\text{mm} \times 1.2\text{mm}$	83	36.6	1470	1803.9	22.7	
sczs2-1-4	120mm $\times$ 5.86mm	321.1	$\Phi 80\text{mm} \times 1.0\text{mm}$	80	35.2	1460	1795.6	23	
sczs2-1-1	120mm $\times$ 5.86mm	321.1	$\Phi 80\text{mm} \times 1.0\text{mm}$	80	20.1	1176	1437.0	22.2	
CFRT40-3	200mm $\times$ 5mm	366	$\Phi 140\text{mm} \times 1.7\text{mm}$	82	24.7	2016	2776.4	37.7	
CFRT40-5	200mm $\times$ 5mm	366	$\Phi 140\text{mm} \times 1.7\text{mm}$	82	32.5	2468	3179.4	28.8	Lu et al. (1999)
CFRT60-4	300mm $\times$ 5mm	366	$\Phi 210\text{mm} \times 2.5\text{mm}$	84	28.3	4603	5616.7	22.0	
CFRT60-5	300mm $\times$ 5mm	366	$\Phi 210\text{mm} \times 2.5\text{mm}$	84	32.5	4381	6065.3	38.4	

CFDST are basically more than 1.2 times of the original results of the pure CFST columns. Therefore, it can be found that with the same cross section, the bearing capacity of CFDST columns is much higher than the original CFST columns. So, it is very applicable to use the inner circular tube to strengthen the CFST column, and the use of CFDST columns can result in significant savings in column size, which ultimately can lead to exert the potentialities of material and thus realize greatly economic saves and higher bearing capacity.

2.6 Summary

The test on CFDST stub columns was introduced to analyze the behavior under axial compressive loading. The typical failure mode of the CFDST column was the local bulging, similar as the square CFST column. Tests confirmed CFDST has the feasibility in theoretical researches and engineering practices as one type of reinforced CFSTs.

Two methods based on the Unified Theory of CFST and elastoplastic limit equilibrium method have been applied to investigate the axial bearing capacity of reinforced CFST stub columns. Besides, one equivalent confinement coefficient was put forward to evaluate the composite performance, so the Unified Theory of CFST was extended to calculate the compressive strength for solid or hollow, circular or non-circular and other various reinforced CFST. In comparison with corresponding test values, there was good agreement between theoretical and experimental results, so it provides a valuable and simple tool for an analysis of the reinforced CFSTs. These two calculated results both have good agreement with the test results. Through data analysis, the study confirmed the ultimate strength calculation results of the limit equilibrium method were found to be more accurate and reliable than the Unified Theory of CFST. But the equivalent confinement coefficient extended from the Unified Theory of CFST is one important parameter to analyze composite performance of reinforced CFSTs. It can not only reflect the physical composition and geometric properties but also mainly reflect the confinement to the concrete in reinforced CFST structures.

CFDST was testified to be the more applicable CFST structure by comparison of mechanical properties and steel ratio. And with consideration of the connection with frame beams, the section of circle inside and square outside was considered as more ideal one among all the various sections of reinforced CFST columns. An optimal design was obtained for CFDST columns by parameter optimization according to the axial behaviors' analysis. The model is extended to predict the optimal design of the inner tube, namely D_i/t_i and D_i/B . Considering the confinement of both outer and inner steel tubes to concrete by limiting its volume expansion, one simplified model was also derived to predict the optimal diameter of circular inner steel tubes.

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Chapter 3

Compressive Stiffness of CFDST Columns



Abstract Columns are more critical in structural frames in earthquake regions, they must exhibit good ductility during an earthquake (Kenji et al. in J Struct Eng 130:180–188, 2004). Based on achievements of CFST columns, this chapter studies the compressive stiffness of CFDST columns. The stiffness is an important property of structural members. The element stiffness matrix is the simple superposition of the material stiffness matrix and geometrical stiffness matrix. The ultimate capacity of columns is influenced by the decrease of column stiffness due to the effect of stability and materials nonlinearity. Expressions of these properties are provided in the AISC 2005 design specification for steel components and American Concrete Institute (ACI) specification for concrete members. However, there is not a unified design code for composite members, and these current design specifications [e.g., AISC Specification in load and resistance factor design specification for structural steel buildings. American Institute of Steel Construction, Chicago, 2005; American Concrete Institute (ACI) (Acceptance criteria for moment frames based on structural testing and commentary. American Certification Institute, Farmington Hills, 2014)] provide different design expressions for CFST components. Herein, the current specification provisions are reviewed and compared. Differences and inconsistencies between these provisions are noted. Axial compression stiffness of CFDST can be obtained by the Unified Theory of CFST, which is testified by a theoretical analysis of method of elasticity.

3.1 Compressive Stiffness of CFST

Since the stiffness has different influences on the calculation result of internal force, deformation and stability, it is advisable to calculate that of steel and concrete separately in CFST. The methods recommended by some different country codes and relevant research literatures can be described as following.

- (1) A calculation method in consideration of a reduction coefficient was adopted in the *Specification for Design and Construction of CFST Structures* (JCJ 01-89) (1989) published by the state bureau of building materials industry in China. The elastic modulus E_{sc} of the CFST column is calculated by

$$E_{sc} = 0.85[(1 - \alpha)E_c + \alpha E_s] \quad (3.1)$$

where E_c is the elastic modulus of concrete; E_s is the elastic modulus of steel; α is the steel ratio of CFST. Then, the compression stiffness can be described by $E_{sc}(A_s + A_c)$; A_s , A_c are the area of the steel and concrete respectively.

- (2) *Specification for Design and Construction of CFST Structures* (CECS 28: 90), China. Composite stiffness is calculated without consideration of the reduction coefficient. The elastic modulus of concrete under tension and compression is determined according to the *Code for Design of Concrete Structures*. Both the compressive stiffness and bending stiffness of CFST are obtained from simple superposition of the corresponding stiffness of the steel tube and concrete. AIJ code (1997), British code BS 5400 (1999), *Engineering Construction Standard* (DB 29-57-2003), Tianjin, China, *Technical Specification for Steel Reinforced Concrete Composite Structures* (JGJ 138-2001) etc. also use this method to calculate the compression stiffness as following.

$$EA_{sc} = E_s A_s + E_c A_c \quad (3.2)$$

So, the elastic modulus of CFST is:

$$E = (E_s A_s + E_c A_c) / A_{sc} \quad (3.3)$$

- (3) In AISC-LRFD (1999), American Institute of Steel Construction does not consider the contribution of concrete to the geometric characteristics (such as area and moment of inertia), but does in terms of stiffness and other mechanical properties (such as strength and elastic modulus). In this code, the idea of establishing the design formula for bearing capacity and moment of inertia of CFST is to consider the outer steel tube together with the contribution of concrete inside the tube. Then the elastic modulus of the composite section is defined and derived by

$$E_{sc} = E_s + 0.4E_c \frac{A_c}{A_s} \quad (3.4)$$

- (4) National military standard GJB4142-2000, China, Fujian province engineering construction standard *Technical Specification for Concrete-filled Steel Tubular Structures* DBJ 13-51-2003 (2003), and China electric power industry standard DL/T5085-1999 (1999) adopt the combined axial compressive stiffness at the basis of the Unified Theory of CFST. Through regression analyses of numerical calculation results in the literature (Zhong 1999, 2000), a simplified calculation formula of combined modulus was given, which was the ratio of nominal normal stress to corresponding positive strain as follows.

$$E_{sc} = \frac{f_{sc}^p}{\varepsilon_{sc}^p} \quad (3.5)$$

where ε_{sc}^p is the nominal composite proportional limit strain of CFST; f_{sc}^p is the nominal composite proportional limit stress.

In conclusion, compressive stiffness of CFST is obtained mostly by superposition calculation of concrete and steel or by reducing the stiffness of concrete. However, it is not solved how the value of compressive stiffness of CFST affects the calculation results of internal force, deformation and stability. The additional internal force will be obtained due to the larger stiffness in statically indeterminate structures. Therefore, the stiffness should be taken as the large value calculated by the superposition method when calculating internal force, and it should be taken as the small value calculated by reducing concrete's contribution when calculating deformation and stability (Ma et al. 2004).

3.2 Simple Superposition Method

Similar to compressive stiffness calculation of CFST, the composite axial stiffness of CFDST can also be calculated by simple superposition to add corresponding stiffness of steel and concrete together without considering the interaction between steel tube and core concrete, i.e.

$$E_o A = E_c A_c + \sum E_s A_s \quad (3.6)$$

Then the elastic modulus of CFDST is defined and derived by

$$E_o = \frac{E_c A_c + \sum E_s A_s}{A} \quad (3.7)$$

where E_o is the elastic modulus of CFDST using simple superposition, A is the area of the whole composite column. In fact, there is confining force between the concrete and steel tubes of the CFDST column under axial compression. But the stiffness improvements of concrete under three-dimensional compression are not considered in the composite compressive stiffness by the superposition method, so the calculation value should be relatively smaller.

3.3 Calculation by Unified Theory of CFST

As described in Sect. 2.4, using the method of the Unified Theory of CFST, the whole cross-section will be considered as a unified body or a new composite material to study the ultimate bearing capacity for reinforced CFSTs. It can be concluded that the equivalent confinement coefficient is one important parameter to analyze mechanical performance of the composite column. It can not only reflect the physical composition

and geometric properties but also mainly reflect the confinement to the concrete. Although the concrete may have been cracked or crushed when the axial load attains a certain level, the CFST column under compression may still has high ductility and energy dissipation ability since the concrete is confined well (Kuranovas et al. 2009). Actually, effective confined concrete can change its failure mode from a brittle one to a relatively ductile one. And this working mechanism is also suitable for the CFDST because this confining effect to concrete is strengthened by double steel tubes. It can also be applied to the calculation of stiffness according to the geometric characteristics of the whole section. The combined modulus in the elastic phase can be obtained from the stress-strain relation, and the formula can be calculated as

$$E_{ssc} = \frac{f_{ssc}^p}{\varepsilon_{ssc}^p} = \frac{(0.192 f_{ss}/235 + 0.488)}{0.67 f_{ss}} f_{ssc} E_s \quad (3.8)$$

where f_{ssc}^p is the proportional limit stress of CFDST column; ε_{ssc}^p is the proportional limit strain of CFDST column; f_{ssc} is the composite compressive strength of CFDST column (see Eq. 2.18); f_{ss} is the weighted average yield strength of steel tubes, $f_{ss} = \frac{\sum A_s f_s}{\sum A_s}$. Then the axial compressive stiffness is $EA = E_{ssc}(A_s + A_c)$. In order to testify the accuracy of the stiffness obtained by the Unified Theory of CFST, stiffness of CFDST columns under small displacement was analyzed by the energy method. The modified elastic modulus of the concrete and steel tubes were deduced respectively to calculate the composite stiffness of the CFDST stub column.

3.4 Elasticity Solutions

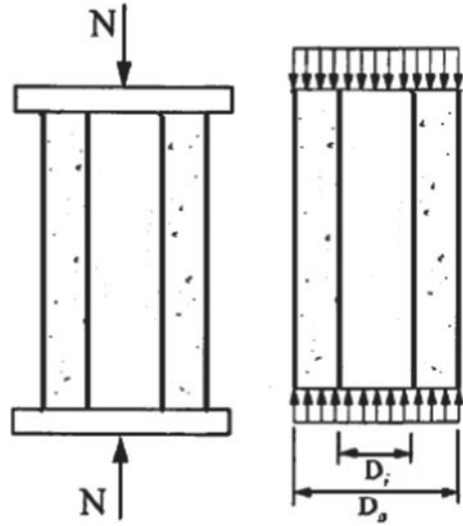
3.4.1 Basic Model

Take the example of hollow CFDST with an inner circular-outer circular section (Type *j* in Fig. 1.10), the loading and calculation model of the stub column are shown in Fig. 3.1. Then, composite stiffness can be obtained by the sum of every component of the stub column (Tan and Zhang 2010). There are sandwich concrete, outer and inner steel tubes. The superposition method is used simply to estimate the composite stiffness without consideration of the interaction between concrete and steel tubes. The expression is

$$E_o A = E_s A_{si} + E_s A_{so} + E_c A_c \quad (3.9)$$

Actually, each elastic modulus has been influenced because there is confining force between concrete and steel tubes. During the process of compression of hollow CFDST columns, there exists horizontal deformation when vertical deformation happened upon the whole section. Horizontal deformation coefficient v_c is the ratio of horizontal deformation to vertical deformation in the calculation of compressive

Fig. 3.1 Loading model and calculating model



concrete. The steel's Poisson's ratio ν_s remains essentially the same value as 0.283, while the concrete's horizontal deformation coefficient ν_c is 0.167 at the elastic stage. With the increasing axial compression, ν_c starts to gradually increase then exceeds ν_s . Because there are many micro-cracks in concrete, ν_c increases gradually till close to 0.5 before the column enters the plastic stage (Zhong 1996). However, both outer and inner steel tubes were confining the sandwich concrete to postpone its expansion. Therefore, we can quantitatively analyze how much the elastic modulus is influenced by this interaction. The concrete can be regarded as a thick-walled cylinder with the inside and outside pressure coming from the inner and outer tubes, while the steel tubes can be regarded as thin-walled cylinders. Defining the changed elastic modulus as modified elastic modulus, then Eq. 3.1 can be changed as

$$E_{ssc}A = \bar{E}_{si}A_{si} + \bar{E}_{so}A_{so} + \bar{E}_cA_c \quad (3.10)$$

That means compressive stiffness can be calculated by modifying the elastic modulus. And the modified elastic modulus ($\bar{E}_{si}$, $\bar{E}_{so}$, $\bar{E}_c$) can be deduced by energy theory.

3.4.2 Energy Theory

Basic assumptions: (1) the radial deformation of concrete and tube conforms to the deformation compatibility condition and the plane cross-section assumption without considering the effects of shear deformation; (2) confining force p of concrete under lateral compression is proportional to the lateral deformation u_r , i.e. $p = ku_r$; (3) the pressures from inner and outer steel tubes to inside concrete are equal within the

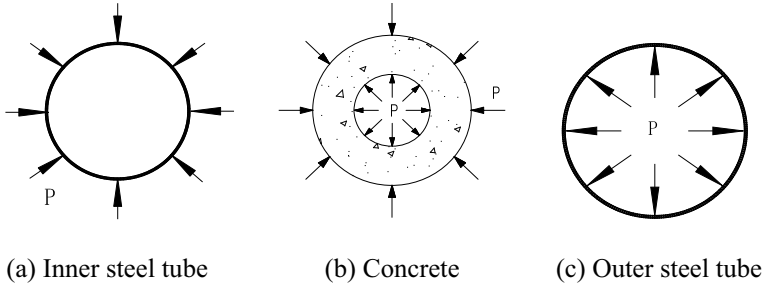


Fig. 3.2 Simplified stress model of every component of hollow CFDST

elastic range. Every part of hollow CFDST column can be analyzed respectively as shown in Fig. 3.2.

Under the axial compression N , the column has a small axial deformation as $-u_z$ ('-' means compression), meanwhile, concrete expands both inward and outward. The absolute lateral deformation is considered as u_r . For each unit length of the column, the axial strain ε_z is equaled to u_z , if the loading process can be considered as a slow quasi-static deformation mode, the strain energy will be

$$U = \frac{1}{2} \int_V \{\sigma\} \{\varepsilon\}^T dV \quad (3.11)$$

According to the Hooke's law, there is

$$\begin{aligned} \varepsilon_r &= \frac{\sigma_r}{E} - \nu \left(\frac{\sigma_\theta}{E} + \frac{\sigma_z}{E} \right) \\ \varepsilon_\theta &= \frac{\sigma_\theta}{E} - \nu \left(\frac{\sigma_r}{E} + \frac{\sigma_z}{E} \right) \\ \varepsilon_z &= \frac{\sigma_z}{E} - \nu \left(\frac{\sigma_r}{E} + \frac{\sigma_\theta}{E} \right) \end{aligned} \quad (3.12)$$

Therefore,

$$U = \frac{1}{2} \int_V \{\sigma\} \{\varepsilon\}^T dV = \frac{A}{2E} [\sigma_r^2 + \sigma_\theta^2 + \sigma_z^2 - 2\nu(\sigma_r\sigma_\theta + \sigma_r\sigma_z + \sigma_z\sigma_\theta)] \quad (3.13)$$

Axial compressive force and lateral confining force can be taken as the external forces for every component, so external work is

$$W = \frac{1}{2} N u_z + \frac{1}{2} \oint_S p u_r ds \quad (3.14)$$

Then the potential energy is

$$\Pi = U - W \quad (3.15)$$

Firstly, for the inner steel tube, its stress state can be expressed as

$$\sigma_{si\theta} = -p\lambda_i; \sigma_{sir} = 0; \sigma_{siz} = -\varepsilon_z \bar{E}_{si} \quad (3.16)$$

where $\lambda_i = D_i/2t_i$. According to the generalized Hooke's law, radial strain of the inner steel tube $\varepsilon_{ri} = \frac{\nu_s}{E_s}(p\lambda_i + \bar{E}_{si}\varepsilon_z)$. So, lateral deformation $u_{ri} = \frac{t_i\nu_s}{E_s}(p\lambda_i + \bar{E}_{si}u_z)$. Strain energy of inner steel tube can be obtained by Eq. 3.13.

$$\begin{aligned} U_{si} &= \frac{A_s}{2} \{\sigma\} \{\varepsilon\}^T = \frac{A_{si}}{2E_s} [\sigma_{si\theta}^2 + \sigma_{siz}^2 - 2\nu_s \sigma_{siz} \sigma_{si\theta}] \\ &= \frac{A_{si}}{2E_s} [p^2 \lambda_i^2 + (u_z \bar{E}_{si})^2 - 2\nu_s p \lambda_i (u_z \bar{E}_{si})] \end{aligned} \quad (3.17)$$

External work of inner steel tube can be obtained by Eq. 3.14.

$$W_{si} = \frac{1}{2} N(-u_z) + \frac{1}{2} \oint_s (-p) u_{ri} ds = \frac{1}{2} \bar{E}_{si} A_{si} u_z^2 - \frac{1}{2} \pi D_i p u_{ri} \quad (3.18)$$

Potential energy per unit length can be calculated from Eq. 3.15.

$$\begin{aligned} \Pi_{si} &= U_{si} - W_{si} = \frac{A_{si}}{2E_s} [k^2 u_r^2 \lambda_i^2 + (u_z \bar{E}_{si})^2 - 2\nu_s k u_r \lambda_i (u_z \bar{E}_{si})] - \frac{1}{2} \bar{E}_{si} A_{si} u_z^2 \\ &\quad + \frac{1}{2} \pi D_i k u_r \frac{t_i \nu_s}{E_s} (k u_r \lambda_i + \bar{E}_{si} u_z) \end{aligned}$$

Because the inner tube is kept in a equilibrium state, according to the theorem of stationary value of potential energy, there is $\frac{\partial \Pi_{si}}{\partial u_z} = 0$. That means

$$\frac{A_{si}}{2E_s} (2\bar{E}_{si}^2 u_z - 2\nu_s k u_r \lambda_i \bar{E}_{si}) - \bar{E}_{si} A_{si} u_z + \frac{1}{2} \pi D_i k u_r \frac{t_i \nu_s}{E_s} \bar{E}_{si} = 0 \quad (3.19)$$

Therefore, modified elastic modulus of inner steel tube can be obtained

$$\bar{E}_{si} = E_s + \frac{k\nu_s(D_i - t_i)}{2t_i} \cdot \frac{u_r}{u_z} \quad (3.20)$$

In the above formula, $\frac{u_r}{u_z}$ is the deformation ratio of concrete. They can be solved in the following calculation.

Secondly, for the outer steel tube, its stress state can be expressed as

$$\sigma_{so\theta} = p\lambda_o; \sigma_{sir} = 0; \sigma_{soz} = -\varepsilon_z \bar{E}_{so} \quad (3.21)$$

where $\lambda_o = D_o/2t_o$. According to the generalized Hooke's law, radial strain of the outer steel tube $\varepsilon_{ro} = -\frac{\nu_s}{E_s}(p\lambda_o - \bar{E}_{so}\varepsilon_z)$. So, lateral deformation $u_{ro} = -\frac{t_o\nu_s}{E_s}(p\lambda_o - \bar{E}_{so}u_z)$. Strain energy of outer steel tube can be obtained by

$$\begin{aligned} U_{so} &= \frac{A_{so}}{2}\{\sigma\}\{\varepsilon\}^T = \frac{A_{so}}{2E_s}[\sigma_{so\theta}^2 + \sigma_{soz}^2 - 2\nu_s\sigma_{soz}\sigma_{so\theta}] \\ &= \frac{A_{so}}{2E_s}[p^2\lambda_o^2 + (u_z\bar{E}_{so})^2 + 2\nu_s p\lambda_o(u_z\bar{E}_{so})] \end{aligned} \quad (3.22)$$

External work of outer steel tube can be obtained by

$$W_{so} = \frac{1}{2}N(-u_z) + \frac{1}{2}\oint_s (-p)u_{ro}ds = \frac{1}{2}\bar{E}_{so}A_{so}u_z^2 - \frac{1}{2}\pi D_o p u_{ro} \quad (3.23)$$

Potential energy of outer steel tube per unit length is

$$\begin{aligned} \Pi_{so} &= U_{so} - W_{so} = \frac{A_{so}}{2E_s}[p^2\lambda_o^2 + (u_z\bar{E}_{so})^2 + 2\nu_s p\lambda_o(u_z\bar{E}_{so})] \\ &\quad - \frac{1}{2}\bar{E}_{so}A_{so}u_z^2 + \frac{1}{2}\pi D_o p u_{ro} \\ &= \frac{A_{so}}{2E_s}[k^2u_r^2\lambda_o^2 + (u_z\bar{E}_{so})^2 + 2\nu_s k u_r \lambda_o(u_z\bar{E}_{so})] \\ &\quad - \frac{1}{2}\bar{E}_{so}A_{so}u_z^2 - \frac{1}{2}\pi D_o k u_r \frac{t_o\nu_s}{E_s}(k u_r \lambda_o - \bar{E}_{so}u_z) \end{aligned} \quad (3.24)$$

With the same method as the inner tube, we can get modified elastic modulus of the outer steel tube

$$\bar{E}_{so} = E_s - \frac{k u_r \nu_s (D_o + t_o)}{2t_o u_z} \quad (3.25)$$

Lastly, for the concrete, its stresses can be obtained by the famous Lamé formula.

$$\begin{aligned} \sigma_{cr} &= \frac{-D_i^2}{D_o^2 - D_i^2}(1 - \frac{D_o^2}{4r^2})p - \frac{D_o^2}{D_o^2 - D_i^2}(1 - \frac{D_i^2}{4r^2})p = -p \\ \sigma_{c\theta} &= \frac{-D_i^2}{D_o^2 - D_i^2}(1 + \frac{D_o^2}{4r^2})p - \frac{D_o^2}{D_o^2 - D_i^2}(1 + \frac{D_i^2}{4r^2})p = -p \\ \sigma_{cz} &= -u_z\bar{E}_c \end{aligned} \quad (3.26)$$

Strain energy of concrete can be obtained by

$$\begin{aligned}
 U_c &= \frac{A_c}{2} \{\sigma\} \{\varepsilon\}^T = \frac{A_c}{2E_c} [\sigma_{cr}^2 + \sigma_{c\theta}^2 + \sigma_{cz}^2 - 2\nu_c(\sigma_{cr}\sigma_{c\theta} + \sigma_{cr}\sigma_{cz} + \sigma_{cz}\sigma_{c\theta})] \\
 &= \frac{A_c}{2E_c} [2p^2 + (\varepsilon_z \bar{E}_c)^2 - 2\nu_c(p^2 + 2p\varepsilon_z \bar{E}_c)] \quad (3.27)
 \end{aligned}$$

external work of concrete can be obtained by

$$\begin{aligned}
 W_c &= \frac{1}{2} N(-u_z) + \frac{1}{2} \oint_s (-p) u_{ro} ds + \frac{1}{2} \oint_s (-p) u_{ri} ds \\
 &= \frac{1}{2} \bar{E}_c A_c u_z^2 - \frac{1}{2} \pi D_o p u_{ro} - \frac{1}{2} \pi D_i p u_{ri} \quad (3.28)
 \end{aligned}$$

Potential energy of concrete per unit length will be

$$\begin{aligned}
 \Pi_c &= U_c - W_c = \frac{A_c}{2E_c} [2p^2 + (u_z \bar{E}_c)^2 - 2\nu_c(p^2 + 2p u_z \bar{E}_c)] \\
 &\quad - \frac{1}{2} \bar{E}_c A_c u_z^2 + \frac{1}{2} \pi D_o p u_{ro} + \frac{1}{2} \pi D_i p u_{ri} \\
 &= \frac{A_c}{2E_c} [2k^2 u_r^2 + (u_z \bar{E}_c)^2 - 2\nu_c(k^2 u_r^2 + 2k u_r u_z \bar{E}_c)] \\
 &\quad - \frac{A_c}{2} \bar{E}_c u_z^2 - \frac{t_o \nu_s}{2E_s} \pi D_o (k^2 u_r^2 \lambda_o - k \bar{E}_{so} u_r u_z) \\
 &\quad + \frac{t_i \nu_s}{2E_s} \pi D_i (k^2 u_r^2 \lambda_i + k \bar{E}_{si} u_r u_z) \quad (3.29)
 \end{aligned}$$

According to the theorem of stationary value of potential energy, there is $\frac{\partial \Pi_c}{\partial u_z} = 0$ and $\frac{\partial \Pi_c}{\partial u_r} = 0$

$$\begin{aligned}
 &\frac{A_c}{2E_c} \left(2\bar{E}_c^2 u_z - 4\nu_c k u_r \bar{E}_c \right) - A_c \bar{E}_c u_z + \frac{t_o \nu_s}{2E_s} \pi D_o k u_r \bar{E}_{so} \\
 &\quad + \frac{t_i \nu_s}{2E_s} \pi D_i k u_r \bar{E}_{si} = 0 \quad (3.30)
 \end{aligned}$$

Modified elastic modulus of concrete can be obtained as follow.

$$\bar{E}_c = E_c + \frac{k u_r (4E_s \bar{E}_c A_c \nu_c - E_c \bar{E}_{so} \nu_s \pi D_o t_o - E_c \bar{E}_{si} \nu_s \pi D_i t_i)}{2u_z E_s \bar{E}_c A_c} \quad (3.31)$$

Compared with other parameters, E_s , $\bar{E}_c$, $\bar{E}_{so}$, $\bar{E}_{si}$, E_c are all much larger ones. So just set: $E_s \bar{E}_c = E_c \bar{E}_{so} = E_c \bar{E}_{si} = E_s E_c$

$$\bar{E}_c = E_c + \frac{k u_r (4A_c \nu_c - \nu_s A_{so} - \nu_s A_{si})}{2u_z A_c} \quad (3.32)$$

$$\begin{aligned}
\frac{\partial \Pi_c}{\partial u_r} &= 0 \text{ i.e.} \\
\frac{A_c}{2\bar{E}_c} [4k^2 u_r - 2v_c(2k^2 u_r + 2k u_z \bar{E}_c)] - \frac{t_o v_s}{2\bar{E}_s} \pi D_o (2k^2 u_r \lambda_o - k \bar{E}_{so} u_z) + \\
\frac{t_i v_s}{\bar{E}_s} \pi D_i (2k^2 u_r \lambda_i - k \bar{E}_{si} u_z) &= 0 \\
\frac{u_r}{u_z} &= \frac{4E_s \bar{E}_c A_c v_c - E_c \bar{E}_{so} v_s \pi D_o t_o - E_c \bar{E}_{si} v_s \pi D_i t_i}{4k E_s A_c (1 - v_c) - 2k E_c \lambda_o v_s \pi D_o t_o + 2k E_c \lambda_i v_s \pi D_i t_i} \quad (3.33)
\end{aligned}$$

And it can be simplified as

$$\frac{u_r}{u_z} = \frac{1}{k} \cdot \frac{4v_c A_c - v_s (A_{so} + A_{si})}{4A_c \left(\frac{1-v_c}{E_c} - \frac{v_s}{E_s} \right)} \quad (3.34)$$

Therefore, $k \frac{u_r}{u_z} = \frac{4v_c A_c - v_s (A_{so} + A_{si})}{4A_c \left(\frac{1-v_c}{E_c} - \frac{v_s}{E_s} \right)}$. Just substitute these two parameters in the previous expressions of modified elastic modulus, there are

$$\begin{aligned}
\bar{E}_{si} &= E_s + \frac{E_s v_s (D_i - t_i) (4v_c - v_s \alpha)}{8t_i \left(\frac{1-v_c}{k_E} - v_s \right)} \\
\bar{E}_{so} &= E_s - \frac{E_s v_s (D_o + t_o) (4v_c - v_s \alpha)}{8t_o \left(\frac{1-v_c}{k_E} - v_s \right)} \\
\bar{E}_c &= E_c + \frac{E_c (4v_c - v_s \alpha)^2}{8(1 - v_c - v_s k_E)} \quad (3.35)
\end{aligned}$$

where modulus ratio $k_E = \frac{E_c}{E_s}$, diameter ratio $k_D = \frac{D_i}{D_o}$ and steel ratio $\alpha = \frac{A_{so} + A_{si}}{A_c}$.

All these results show that the modified elastic modulus of inner tube and concrete are both improved because these two parts are under three-dimensional compressive stresses, while the modified elastic modulus of outer tube is reduced because it is under tensile and compressive stresses. Substitute the modified elastic modulus into Eq. 3.2, then it can be changed as

$$\begin{aligned}
\bar{E}_{ssc} A &= E_s A_{si} + E_s A_{so} + E_c A_c \\
&+ E_s E_c \frac{4v_c A_c - v_s A_{so} - v_s A_{si}}{8A_c [E_s (1 - v_c) - E_c v_s]} \left[4v_c A_c + \frac{v_s (D_i - 2t_i) A_{si}}{t_i} - \frac{v_s (D_o + 2t_o) A_{so}}{t_o} \right] \quad (3.36)
\end{aligned}$$

Because steel tubes in this book are considered as thin-walled cylinders, the above formula can be transformed as

$$\bar{E}_{ssc} A = E_s A_{si} + E_s A_{so} + E_c A_c + E_s E_c (v_c - v_s) \frac{4v_c A_c - v_s A_{so} - v_s A_{si}}{2[E_s (1 - v_c) - E_c v_s]} \quad (3.37)$$

Equation 3.36 presents the compressive stiffness of the stub column considering the confining effect. And the composite elastic modulus can be obtained as well.

$$\bar{E}_{ssc} = \frac{1}{A} [E_s A_{si} + E_s A_{so} + E_c A_c + E_s E_c (v_c - v_s) \frac{4v_c A_c - v_s A_{so} - v_s A_{si}}{2E_s(1 - v_c) - 2E_c v_s}] \quad (3.38)$$

For $A = A_{si} + A_{so} + A_c$; $\alpha = \frac{A_{so} + A_{si}}{A_c}$, and $E_0 = \frac{1}{A} (E_s A_{si} + E_s A_{so} + E_c A_c)$
Therefore,

$$\bar{E}_{ssc} = E_0 + \frac{E_s E_c (v_c - v_s)}{\alpha + 1} \cdot \frac{4v_c - v_s \alpha}{2E_s(1 - v_c) - 2E_c v_s} \quad (3.39)$$

According to the formulas, it is shown that the composite elastic modulus has close relation with steel ratio, Poisson's ratio of steel and horizontal deformation coefficient of concrete. Therefore, the composite stiffness increases with the increase of steel ratio, Poisson's ratio of steel and horizontal deformation coefficient of concrete, but it has no evident change with the increase of concrete strength.

3.5 Verification of Theoretical Calculations

3.5.1 Comparison with Experimental Results

The calculation results of composite elastic modulus E_{ssc} from the Unified Theory of CFST and $\bar{E}_{ssc}$ from the energy variational method are listed in Table 3.1 for hollow CFDST columns. The calculation process of E_{ssc} refers to Eq. 3.8 in Sect. 3.3.4. E_0 is the elastic modulus obtained by the simple superposition method. Among E_0 , E_{ssc} , and $\bar{E}_{ssc}$, E_0 is the smallest since no confinements between double steel tubes and concrete is involved. The mean of $\bar{E}_{ssc}/E_{ssc}$ is 0.973, and the calculation values obtained by the Unified Theory of CFST and energy variational method are very close. Since axial compressive stiffness of the composite column is the product of composite elastic modulus times the whole cross-section area, it is verified feasible to apply these two methods to calculate the axial stiffness. $\bar{E}_{ssc}$ is nearly 1.53–2.72 times of E_c , and 0.25–0.44 times of E_s . Compared with E_0 , composite elastic modulus E_{ssc} , $\bar{E}_{ssc}$ has been increased by nearly 10%. Using the example of hollow CFST, it can be testified that the equivalent confinement coefficient can also be used to calculate composite stiffness of reinforced CFSTs by the Unified Theory of CFST.

Table 3.1 Calculation of elastic modulus of CFDST columns

Specimens	D_o (mm)	D_i (mm)	α	$E_c(10^4 \text{ MPa})$	$E_s(10^5 \text{ MPa})$	$E_0(10^5 \text{ MPa})$	$E_{ssc}(10^4 \text{ MPa})$	$\bar{E}_{ssc}(10^4 \text{ MPa})$	$\bar{E}_{ssc}/E_c$	$\bar{E}_{ssc}/E_s$	References
A1-1	74.8	62	0.318	3.598	1.97	7.480	8.289	7.920	2.201	0.402	Wei et al. (1995)
A2-1	75.4	62.7	0.398	3.598	1.97	8.180	8.847	8.589	2.387	0.436	
B1-1	81	62	0.199	3.598	1.97	6.266	6.626	6.761	1.879	0.343	
C1-1	87.4	61.8	0.147	3.598	1.97	5.661	6.834	6.183	1.718	0.314	
D4-1	99.9	74	0.102	3.598	1.97	5.095	5.933	5.642	1.568	0.286	
D5-1	99.8	61.4	0.064	3.598	1.97	4.572	5.057	5.143	1.429	0.261	Tao et al. (2004)
E4-1	101.4	63.5	0.115	3.598	1.97	5.253	6.011	5.793	1.610	0.294	
E5-1	114.3	76.1	0.151	3.598	1.97	5.708	6.511	6.228	1.731	0.316	
E6-1	114.3	88.9	0.252	3.598	1.97	6.847	7.819	7.515	2.033	0.371	
cc2	180	48	0.091	3.410	2.06	4.842	5.250	5.362	1.573	0.260	
cc3	180	88	0.130	3.410	2.06	5.393	6.009	5.892	1.728	0.286	
cc4	180	140	0.300	3.410	2.06	7.377	8.344	7.797	2.287	0.378	
cc5	114	58	0.214	3.410	2.06	6.443	7.331	6.900	2.024	0.335	
cc6	240	114	0.095	3.410	2.06	4.904	5.421	5.422	1.590	0.263	
cc7	300	165	0.089	3.410	2.06	4.813	5.247	5.334	1.565	0.259	

3.5.2 Parametric Study

Figure 3.3 shows the composite stiffness $\bar{E}_{ssc}$ of two specimens from the references are changed with the horizontal deformation coefficient of concrete ν_c when $\nu_s = 0.283$. When ν_c equals to ν_s , it can be shown that there is no reaction between concrete and steel, so the composite elastic modulus is just the superposed value E_o . When ν_c is smaller than ν_s , the composite elastic modulus has not been decreased obviously, can also be considered to be equal to E_o . Once ν_c is larger than ν_s , the composite elastic modulus will be improved rapidly with the ascending ν_c . Figure 3.4 shows the composite stiffness of both group specimens are changed with the steel ratio when $\nu_s = 0.283$ and $\nu_c = 0.4$. As can be seen, the composite elastic modulus is improved

Fig. 3.3 Relationship between $\bar{E}_{ssc}$ and ν_c

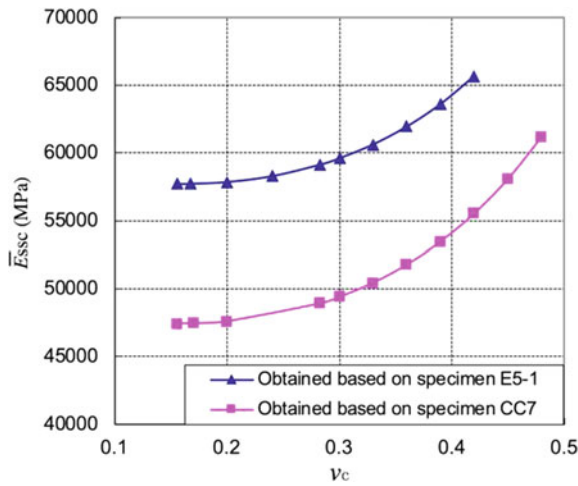
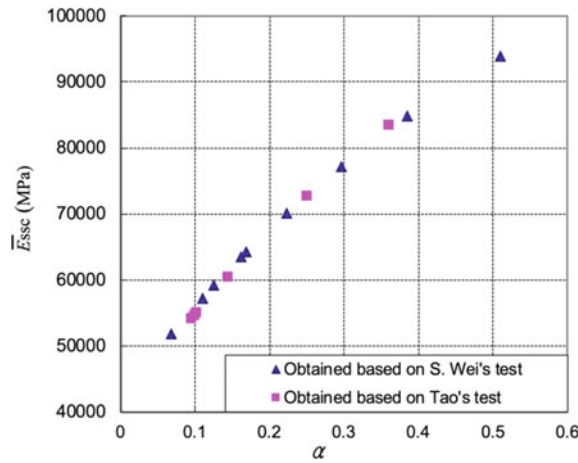


Fig. 3.4 Relationship between $\bar{E}_{ssc}$ and α



nearly linearly as steel ratio increases. So, it can be concluded that the main factors such as the coefficient of lateral deformation of concrete, Poisson's ratio of steel and steel ratio, evidently influenced the axial compression stiffness of the hollow CFDST column.

3.6 Summary

This chapter firstly summarized the stiffness calculation of the CFST column in some codes and literatures from different countries. As for the CFDST column, the composite axial stiffness obtained by the superposition method is smaller because the elastic modulus of the core concrete is not considered to be increased due to the confining effect from steel tubes. Under the guideline of the Unified Theory of CFST, the composite modulus and compressive stiffness were calculated by the equivalent confinement coefficient. Using the example of hollow CFST, this axial stiffness value is testified by energy variational method. Therefore, the equivalent confinement coefficient is also one important parameter to analyze composite stiffness of reinforced CFSTs. It can not only reflect the physical composition and geometric properties but also mainly reflect the confinement to the concrete in CFDST structures.

By comparison, the composite stiffness of hollow CFDST increases with the improvement of steel ratio, Poisson's ratio of steel and horizontal deformation coefficient of concrete, but it has no evident change with the increase of concrete strength.

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Chapter 4

Frame Joint Forms in CFDST Structures



Abstract The CFDST column consists of an inner tube and an outer tube (Type *a, b* in Fig. 1.7), which are both filled with concrete. After discussion of the axial bearing capacity and stiffness, CFDST has been confirmed as the ideal cross-section type among the reinforced CFST. With the superior performance, CFDST has a theoretical significance and application value to be applied in ultra-high-rise structure frames, so the research on various joint forms is a meaningful and worthwhile job to build a solid and authentic foundation for the institution and design code of the CFDST frame structures in the future. The beam-to-column connection is a very important and weak part in the CFST system. They are divided into CFST column-to-reinforced concrete (RC) beam joints and CFST column-to-steel beam joints according to the types of beams. Connections should possess a reliable force transfer mechanism and a convenient construction procedure in the structure design (Han and Zhang 2013; Hortencio and Falcon 2018). For the CFST structure, many researches have been conducted to better understand the behavior of CFST frames, as well as to develop various joint forms that demonstrate adequate structural performance under seismic loading conditions (Bertero et al. 1973; Han and Li 2010; Li et al. 2009; Nie et al. 2008). In Sect. 1.4, we have reviewed common connection types in CFST structures, then some of them can be improved to apply in CFDST structures. Several applicable beam-to-column connections of CFDST frame structures will be introduced in this chapter. The basic construction forms of five frame joints including ring beam joint, external diaphragm joint, vertical stiffener joint, anchored steel beam joint, and bolted T-plate joint, are described respectively as follows. All the joints modeled the interior joint of a moment-resisting frame consisting of CFDST columns and H-shaped steel beams or RC beams (Zhang and Wang 2016; Zhang and Zhou 2019; Zhang et al. 2014). The force transfer mechanism of each beam-to-column connection is described respectively. According to the joint design principle for the CFST structure, there are some suggestions for the frame joint design in the CFDST structure, and some joint components need to be optimized to make joints behave better in the new structure.

4.1 Ring-Beam Joint with Discontinuous Outer Tube

The ring-beam joint is a typical through-beam connection because the beam directly pass through the panel zone and through the discontinuous outer tube. Since there are double tubes in the CFDST column, the outer tube is disconnected and RC beam is horizontally continuous through the joint core. The connection details are shown in Fig. 4.1. In the ring-beam joint, the inner tube is continuous to provide the column with enough load bearing capacity and stability. In order to compensate the decreased stiffness of the composite column, a stiffening ring with multi-layer annular reinforcements and radial stirrups is located outside the joint core. Vertical reinforcements are inserted through the upper and lower outer tubes with enough anchorage length in the concrete in the joint zone. Shear studs are welded on the inside wall of outer tubes to ensure bonding between concrete and discontinuous outer steel tubes. The key of this connection system is that transferring of moments and shearing forces is achieved by continuous beam's longitudinal reinforcements;

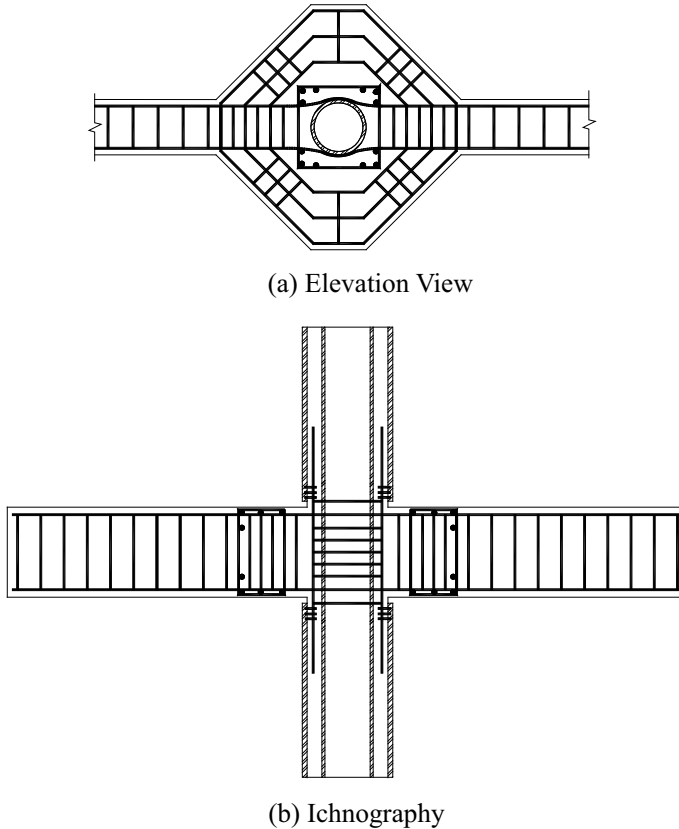


Fig. 4.1 Construction of the ring beam joint

reduced stiffness from the discontinuous outer steel tube can be compensated by the confinement of steel cage anchored inside concrete and the compressive area improvement of the ring beam. Then the outer concrete of the composite column and the ring beam concrete can be casted together to ensure the integrity and continuity of the beam-to-column assembly.

The plastic hinge is predicted to develop at the junction between the stiffening ring and RC beams. However, the vertical reinforcements just insert freely into the discontinuous outer tube so that stress transfer in joint core relies on the inner tube and concrete mainly. It still has some problems to discuss the role of shear studs welded on the inner wall of outer tubes in stress transfer from outer tubes to vertical reinforcements, so welding the vertical reinforcements to the outer tube can make a great function in order to offer a more viable connection for seismic resistant design.

4.2 External Diaphragm Joint

The external diaphragm joint is a common rigid joint for the steel beam in CFST structures. Sometimes, it can also be called as the exterior stiffening ring joint, especially for RC beams as shown in Fig. 4.2. One circular stiffening ring is located outside the CFDST column in the joint zone. The joint can be considered as rigid one because stiffening ring protects the joint panel zone efficiently. According to some FEM results (Quan et al. 2017; Vulcu et al. 2017), the plastic hinge was developed as predicted at the junction between stiffening rings and beams. FEM analysis also showed that the part of stiffening ring at oblique direction between two adjacent beams had a very small stress value, while there was a much larger stress value at the stiffening ring part connected to the beam. So, if the stiffening ring is a steel structure just like the external diaphragm, the shape of the joint can be optimized as the more aesthetic shape as shown in Fig. 4.3. The external diaphragm joint can be

Fig. 4.2 Exterior stiffening ring joint for RC beams

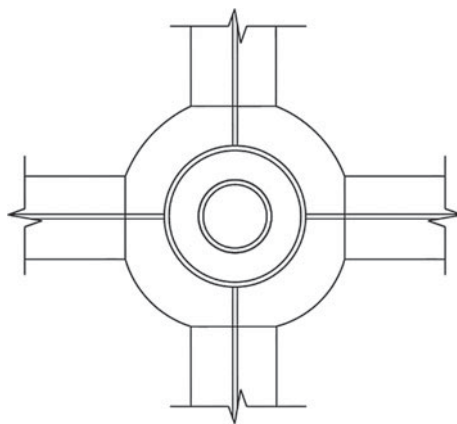
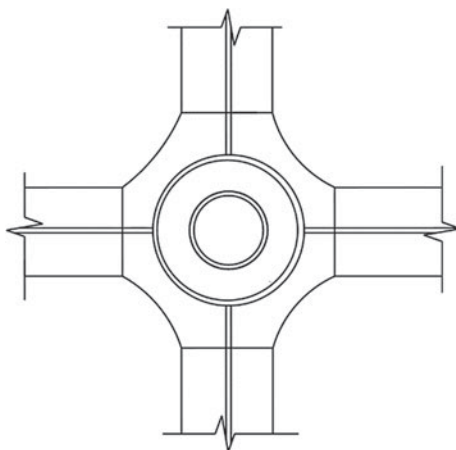


Fig. 4.3 The modified joint for steel beams



modified by adding anchorage plates or adopting a more reasonable aesthetic joint shape. Because the inner tube inside the column has little contribution to the force transfer except it bears the compressive load for granted, and the joint form can be optimized by adding connecting components between inner and outer tubes. The anchorage plate may be horizontal like the penetrating stiffening flange, or be vertical like the penetrating stiffening web. And the anchorage plate can make the external diaphragm and panel zone into an integral whole, which is helpful to improve the joint's mechanical performance.

4.3 Vertical Stiffener Joint

The vertical stiffener joint is proposed as a typical rigid connection for steel beams. In comparison with the vertical stiffener joint in CFST structures, as shown in Fig. 1.5d, anchorage webs can be set between the double tubes in CFDST structures, so vertical stiffeners, anchorage webs, and end plate play the force transfer role together. The transverse profile of the vertical stiffener joint is shown in Fig. 4.4. The anchorage web is groove welded vertically on the inner steel tube, through the outer steel tube,

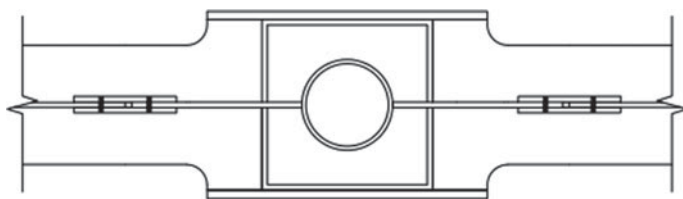


Fig. 4.4 Transverse profile of the vertical stiffener joint

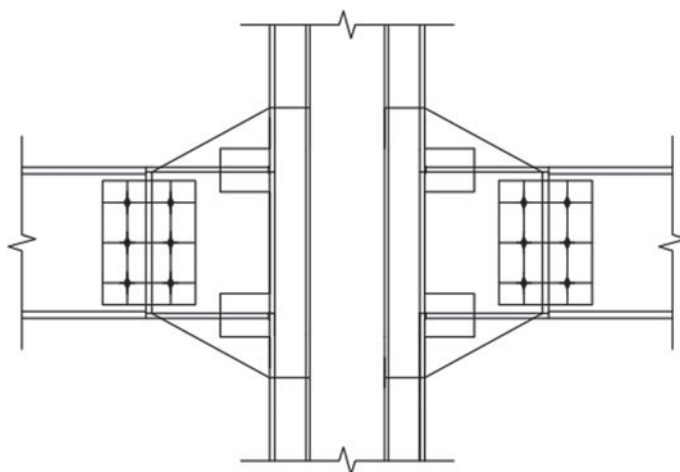


Fig. 4.5 Ribbed vertical stiffener joint

then it connects with the steel beam web by bolts. Meanwhile, the vertical stiffener is groove welded on the column surface, and its overhang welds with the end plate by fillet welds. In order to solve the stress concentration problem, the radius-cut section is used for the end plate. The other side of end plate as wide as the beam is butt welded to the beam flange. In addition, the application of anchorage webs embedded in the concrete between the double tubes, can improve workability and integrity of the joint. And when the anchorage web part inside the double tubes is higher than the steel beam, the anchorage web with triangle rib plates can be chosen as shown in the elevation view (Fig. 4.5). The plastic hinge is predicted at the reduced beam section near the end plate.

4.4 Anchored Steel Beam Joint

The anchored steel beam joint is proposed for steel beams also, as shown in Fig. 4.6. The steel beam is groove welded with both inner and outer steel tubes in this joint construction. In order to ensure concrete is uniformly poured between inner and outer pipe, air holes are reserved on the upper and lower flanges in advance. The attachment weld on inner tube is very important for it is likely to be failed first in practice, but the steel beam is embedded in the concrete, which is helpful to the reliability of the connection. Failure mode in the test (Yan 2011) belonged to shear failure on the column wall, and the panel zone was certainly influenced because the plastic hinge was developed at the junction of the beam and column. So, this joint form can't cater for the seismic requirements of "strong joint & weak member" in structures design. There are three aspects to improve the performance of the anchored steel beam joint.

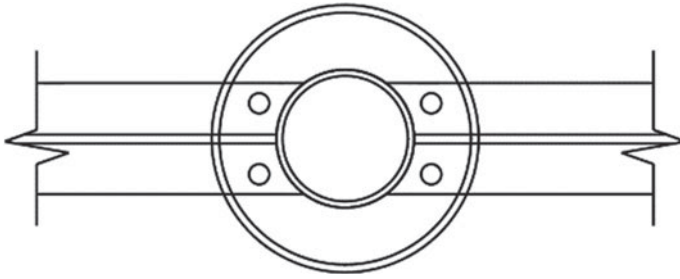


Fig. 4.6 Construction of the anchored steel beam joint

Firstly, the flanges between inner and outer tubes can be widened to improve the joint stiffness and the bonding between steel tube and concrete. And the widened flanges can be extended outside the column for a distance to make plastic hinges far away from the panel zone. Secondly, reduced beam section for the beam near the joint core may be a good choice, i.e. the steel beam flange can be narrowed down at the place where plastic hinge is designed to occur (Wang et al. 2008; Sophianopoulos and Der 2011). Thirdly, anchored triangular ribs may be designed as same as the vertical stiffener joint (see Fig. 4.5) to strengthen the beam-to-column relative rotational stiffness, so this method can move plastic hinges as well. Above all, the anchored steel beam joint should be strengthened to avoid the failure in panel zone.

4.5 High Strength Bolts-T-Plate Joint

The penetrating high strength bolts-T-plate joint was proposed between a CFDST column and an H-shaped steel beam (He et al. 2012). In this joint system, penetrating high strength bolts and T-plates play the force transfer role. As shown in Fig. 4.7, the T-plate flange as an end plate connects with the outer tube in the place of joint zone, and high strength bolts penetrate end plates together with the outer tube before pouring concrete between the double tubes. Meanwhile, T-plate webs connect with steel beam flanges using standard high-strength bolts. The joint initial rotational stiffness is between the rigid connection and the hinge connection, so the joint design belongs to the semi-rigid, weak joint type.

During the tests (Beena et al. 2017; Zhang 2012), each component had different degrees of damage and failure, resulting in comparatively low bearing capacity. For example, the penetrating high strength bolts especially farther away the joint core extended dramatically or nuts slid; nearly all end plates yielded as curved; and parts of high strength bolts connecting the T-plate web and the steel beam flange were broken by shearing force definitely. Therefore, the penetrating high strength bolts-T-plate joint needs a reasonable structure design to prevent brittle failure and to improve the overall bearing capacity and seismic behavior. Since the failure mode is that large beam-to-column relative rotation took place, it is feasible to add a triangular rib

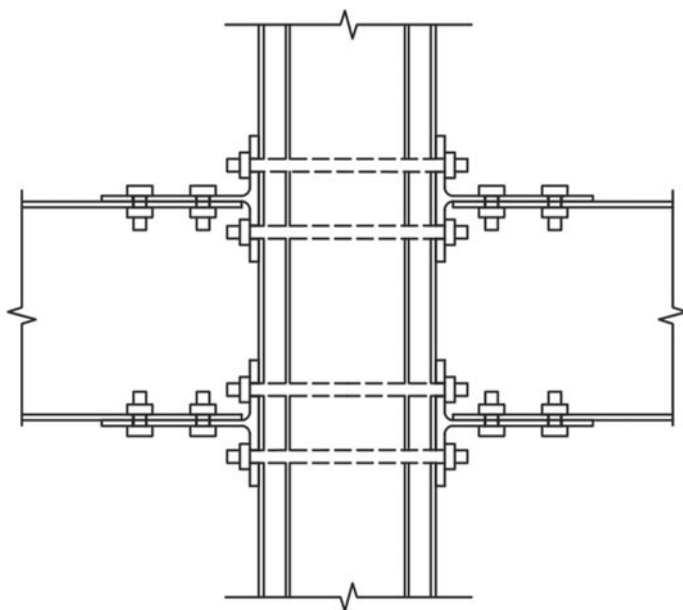


Fig. 4.7 Construction of penetrating high strength bolts-T-plate joint

plate at one side of the T-plate as same as the foregoing ribbed joint, then the T-plate may have enough bending stiffness to strengthen itself, so the outermost bolt can be placed far away from the joint core as possible to reduce its internal force.

4.6 Summary

This chapter focuses on several frame joints, which may be suitable for CFDST columns. The constructional details and force transfer mechanisms of these beam-to-column joints were described. Despite the preliminary achievements on the research of the CFDST frame joint, there is still a long way to meet the requirements proposed by engineering practice. In order to establish a complete research system about the joints design and mechanical performance of the CFDST frame structures as well as propelling its applications. It is necessary to carry out more tests and do the relevant finite element analysis so as to better understand the mechanical performance of these joints. Every beam-to-column connection needs to be designed with suitable construction and parameters, and next investigations may be carried out as following.

For the ring beam joint connected to RC beams, it can be modified by welding vertical reinforcements to outer tube in order to offer a more viable connection for seismic design; for the vertical stiffener joint connected to steel beams, there

need to increase local thickness of steel tube at the place of rib plates, and high-quality welding procedures is also very necessary to ensure the ductility; the external diaphragm joint can be modified by adding anchorage webs or adopting a more reasonable aesthetic joint shape; the anchored steel beam joint can be modified by widening interior flanges between double tubes, or reducing beam section, or using triangular rib plates to make plastic hinges away from the panel zone; for the penetrating high strength bolts-end plate joint, it can be modified by adding triangular rib plates or heightening T-plates.

According to preliminary previews of these beam-to-column connections, the similar types of joints to CFDST columns will be analyzed thoroughly in the following five chapters. All the CFDST columns are chosen as the section of inner circular-outer square (Type *b* in Fig. 1.7) in consideration with the convenient connection construction between square steel tubes and frame beams.

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Chapter 5

Ring Beam Joints to RC Beams

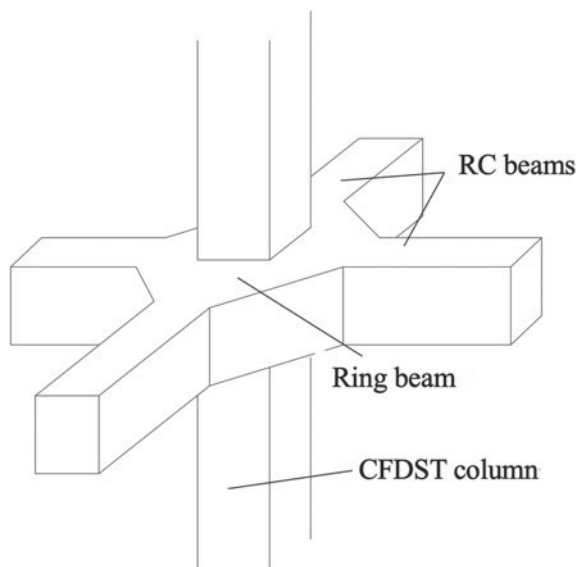


Abstract As we all know that there was severe damage of beam-to-column connections especially in the regions with high seismicity, such as Northridge of the United States, Hanshin of Japan and Wenchuan of China (Popov et al in Eng Struct 20(12):1030–1038 1998; Miller in Eng Struct 20(4–6):249–260, 1998). It is very necessary to improve the bending and compressive resistance of the columns to enhance the ductility and energy dissipation capacity of the connection, which will avoid the brittle fracture. Since the reinforced concrete beam-slab structure is applied in the majority of Chinese buildings, to solve the problem of connection between CFST and RC beams is one of the keys to applying CFST structures. This chapter presents ring beam joints with a discontinuous outer tube between CFDST columns and RC beams, which can be considered as a typical through-beam connection. The behavior of the joints under seismic loads was studied through testing four beam-to-column assemblage specimens. Failure modes, damage mechanism, hysteresis loops, skeleton curves, displacement ductility, stiffness degradation and energy dissipation capability of the joint under fixed axial force and repeated shear force were analyzed in detail. More consideration was focused on parametric analysis for seismic resistance through a three-dimensional nonlinear finite element analysis by software ANSYS.

5.1 Description of Connection System

The ring beam joint with a discontinuous outer tube was designed between CFDST columns and RC beams (Zhang et al. 2012). Figure 5.1 shows the diagram of a ring-beam connection in a typical structure. In consideration with convenient construction to a frame beam, the cross-section of the CFDST column is chosen as inner circular-outer square (Type *b* in Fig. 1.7). In this connection, the outer tube is interrupted and discontinuous in the joint core to achieve the through-beam joint. The reduced stiffness of the composite column due to interruption of the outer steel tube is compensated by an improvement on compressive area of the ring beam.

Fig. 5.1 Diagram of the ring-beam joint with a discontinuous outer tube



The detailed specimen of the ring beam joint with a discontinuous outer tube is shown in Fig. 5.2. Using the recommendations of the design code for concrete structures (GB50010-2010) (2015), the new connection is devised to cater for the seismic requirements of “strong column & weak beam”. The key of this new connection system is that the continuous RC beams are achieved by interruption of the outer steel tube. The transferring of moments and shear forces can be ensured by longitudinal reinforcements ④ in the RC beam. As can be seen, the inner tube is continuous to provide the column with enough load-bearing capacity and stability, and the beam’s longitudinal reinforcements can be continuous and wrap around the inner tube in the joint zone. Meanwhile, vertical reinforcements ① and multiple lateral reinforcements ②, making up a steel cage, are used to confine the core concrete in the joint zone. To ensure the CFDST column’s continuity at the joint, an octagonal ring beam with multi-layer annular reinforcements ⑥ and radial stirrups ⑦ are located outside the joint core to improve compression area of the column. The outer concrete of the composite column and the concrete of the ring beam form one united entity to provide the joint with good integrity and continuity. Vertical reinforcements with enough anchorage length ① are inserted into the outer tube above and below the joint core. Shear studs ⑧ are welded on the inner wall of the discontinued outer tubes, where is close to the joint zone to ensure a more uniform transfer of stresses and enough bonding between the outer tube and concrete.

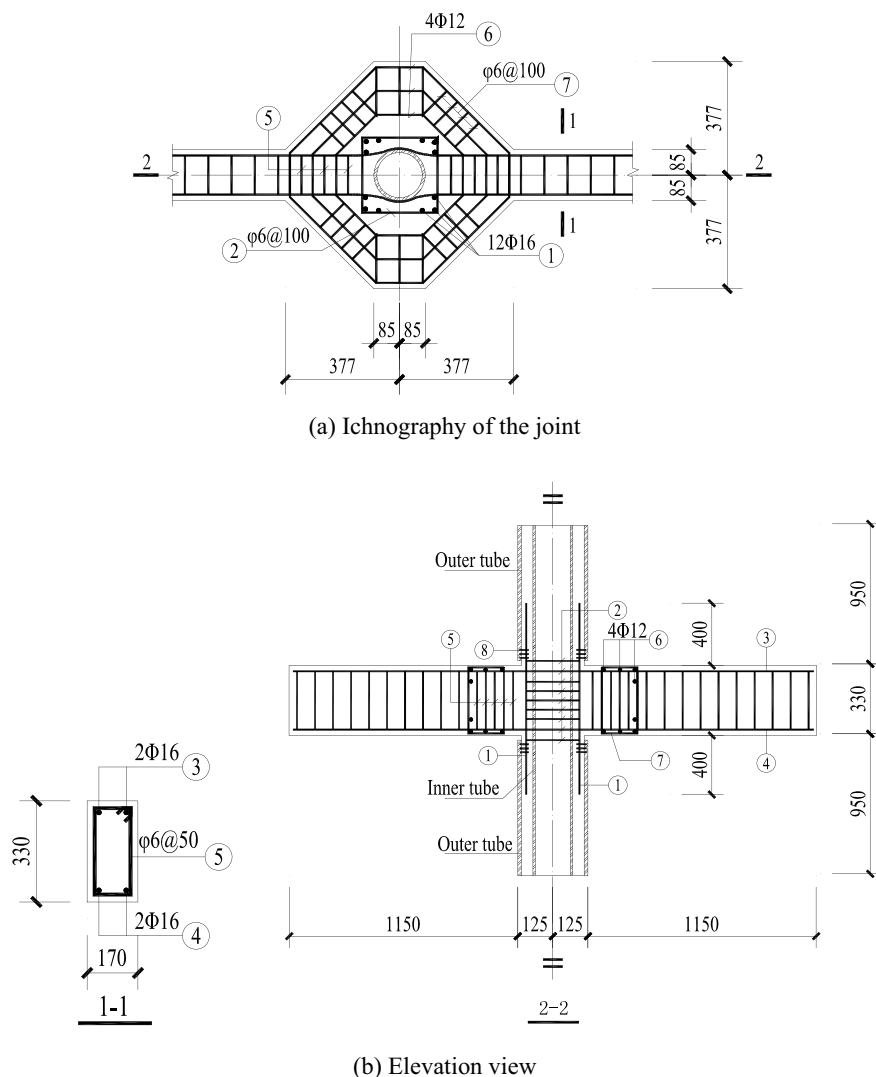


Fig. 5.2 Details of the ring beam joint with a discontinuous outer tube

5.2 Test Specimens and Material Properties

The joint specimens were intended to simulate the connection between an interior column and two adjacent RC beams in a frame structure. Four specimens, named RBJ1-1, RBJ1-2, RBJ2-3 and RBJ2-4 were designed on a 1:2 scale. The parameters include different transverse reinforcement ratios for the ring beam and different diameter for the inner tube are shown in Table 5.1. The steel cage shown in Fig. 5.3

Table 5.1 Parameters of specimens

Specimens	RBJ1-1	RBJ1-2	RBJ2-3	RBJ2-4
Outer tube $\square B \times t$ (side length $\times$ thickness)	$\square 250\text{mm} \times 4\text{mm}$	$\square 250\text{mm} \times 4\text{mm}$	$\square 250\text{mm} \times 4\text{mm}$	$\square 250\text{mm} \times 4\text{mm}$
Inner tube $\Phi 2r_i \times t_i$ (diameter $\times$ thickness)	$\Phi 110\text{mm} \times 3.5\text{mm}$	$\Phi 110\text{mm} \times 3.5\text{mm}$	$\Phi 140\text{mm} \times 3.5\text{mm}$	$\Phi 140\text{mm} \times 3.5\text{mm}$
Volume reinforcement ratio of the ring beam	0.0012	0.0015	0.0012	0.0015

Fig. 5.3 Steel cage and embedded strain gauges

was fabricated and concrete was cast on the construction site while the steel tubes were provided by the manufacturer. Since the outer tube was interrupted at the joint area, there was expected stress concentration at the vicinity of the interruption. Shear studs were welded upon the inner wall of the outer tube as shown in Fig. 5.4. The

Fig. 5.4 Shear studs inside outer tube

Table 5.2 Material properties of steel

Material	Bar Φ 6	Bar Φ 12	Bar Φ 16	Square tube Q235	Circle tube Q215 (Diameter 110 mm)	Circle tube Q215 (Diameter 140 mm)
Yield strength (MPa)	326.5	309.1	330.5	345.9	308.6	326.3
Ultimate strength (MPa)	650.0	584.9	656.5	470.4	465.9	432.5
Elastic modulus (10^5 MPa)	1.83	2.29	2.35	2.03	2.06	2.06

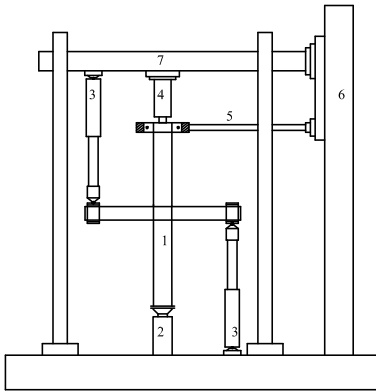
Table 5.3 Material properties of concrete

Specimen number	RBJ1-1	RBJ1-2	RBJ2-3	RBJ2-4
Compressive strength (MPa)	32.5	33.3	30.8	29.7

connection details for RBJ1-2 and RBJ2-4 are shown in Fig. 5.4, and RBJ1-1 and RBJ2-3 have a lower annular rebar ratio for the stiffening ring beam. The steel parameters involved in this test are shown in Table 5.2. The compressive strength of the concrete was determined using concrete cubes with a side length of 150 mm. Each time when pouring concrete, concrete specimens were prepared and cured under the same condition in order to obtain reliable material properties as shown in Table 5.3. The length L and height h of the test specimen are based on the assumption that the point of inflection is at the midspan and mid-height of the prototype beam and column, respectively, where $L = 2550$ mm and $h = 2230$ mm.

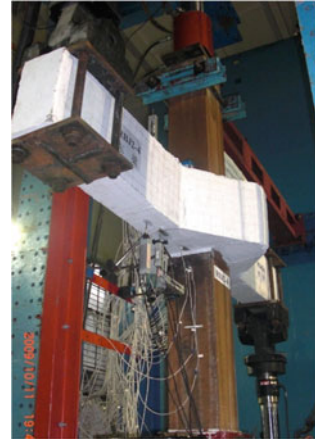
5.3 Test Setup and Measurement Scheme

The test setup is shown in Fig. 5.5. The experiments were conducted in a self-equilibrating reaction frame with a hydraulic jack (capacity of 2,000 kN) at the column top. The column bottom was pinned with a hydraulic jack. The loading mode was controlled by force and displacement successively. Firstly, keep the beam end free. A constant axial compressive force was applied at the column top with a design value controlled by axial compression ratio (0.45 in this test). The axial compression ratio n is a ratio of the axial force N at the column top to the ultimate bearing capacity N_u of the column calculated by the Unified Theory of CFST (see Chap. 2). In order to eliminate influences of initial defects of the specimens, a preloading was carried out at the beginning of test. When the axial load continuously increased to the predetermined value and kept constant for 5 min. Rigid supports and lateral braces



1-Specimen; 2-Steel support; 3-Actuator; 4-Lifting jack;
5-Rigid transverse beam; 6-Reaction wall

(a) Sketch of loading system



(b) Specimen in testing

Fig. 5.5 Test setup of ring beam joint

were installed on two sides of the steel beam. Then, two load actuators (capacity of 500 kN) with roller pads were used to provide vertical cyclic loads at the beam ends.

The strain gauges were preset before casting the concrete to measure radial and tangential strains of the rebars and vertical and transverse strains of the steel tube. The strain gauges for concrete were glued to the concrete surface, as shown in Fig. 5.5b, along with some displacement transducers. The two actuators installed in different directions (pulling and pushing), as shown in Fig. 5.5b, were synchronized to achieve the reversed loads to simulate story displacement due to horizontal loading. Keeping the axial load constant, the reversed cyclic forces at the beam ends were applied until displacement or force limit of the load cell was reached. The load process was firstly controlled with the load increment at the beam end, and each loading step cycled once. After the yielding load was reached, the load steps were controlled by multiples of the yielding displacement, and per amplitude cycled three times until failure of the RC beam.

5.4 Discussion of Test Results

5.4.1 Failure Modes

In general, according to the design criteria of beam failures, the plastic hinge formed as expected at the beam end near the connection, as shown in Fig. 5.6, indicating that



Fig. 5.6 Damage at failure

the design requirement of “strong joints” could be achieved. The cracking development on the wall surface of the ring beam and RC beams was carefully observed and recorded during the loading process. Most cracks were found at the connection zone where the moment is the maximum. The distribution of cracks at the maximum load is shown in Fig. 5.7 where the left part is for the wall surface of the ring beam and the right one is for the RC beam. The cracks on the wall surface of the ring beam developed obliquely up to corner of the column where the weakest confinement existed. Also, the concrete spalling was found on the upper surface of the stiffening ring near

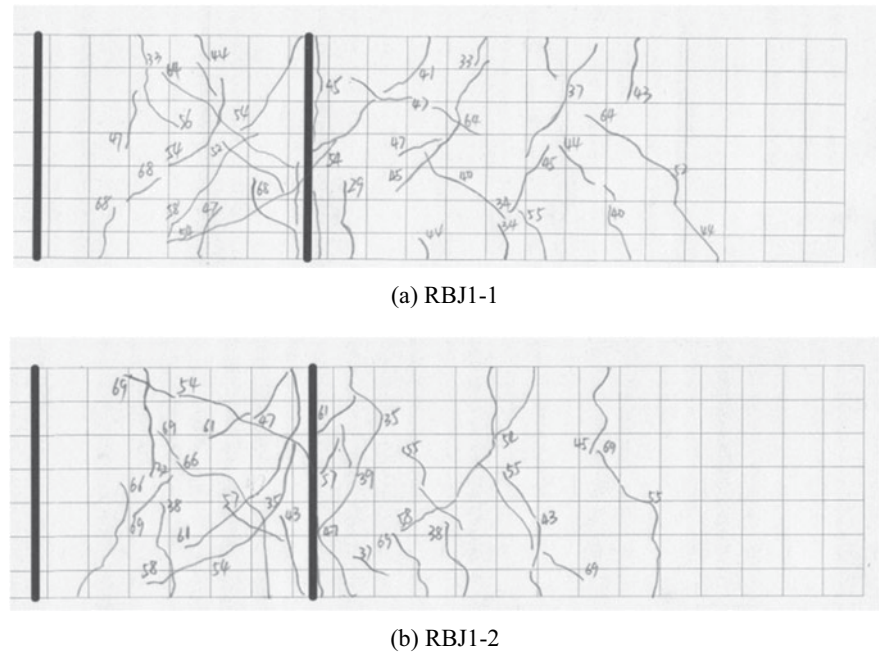


Fig. 5.7 Crack pattern

the beam ends. The yielding of the beam was followed by propagation of cracks at the end of the beam, while the joints and columns maintained a stable status. It was observed from the failure phenomena of the joints that when the reinforcement ratio of the ring beam increased from RBJ-1 with 0.0012 to RBJ1-2 with 0.0015, the position of plastic hinge moved towards the joint core as shown in Fig. 5.6. The main reason is that the ring beam with higher reinforcement ratio can improve the bending stiffness of the RC beam. In general, after the joints failed, no failures phenomena were resulted from the composite column in the joints, even with different reinforcement ratios and different inner steel tubes. Since the joint core concrete basically remained well, and it can be concluded that the steel cage in the joint core and the ring beam make the joint work well with a final ductile damage.

5.4.2 *P-Δ Hysteresis Loops*

The *P-Δ* hysteresis loops of the four specimens are shown in Fig. 5.8, where the vertical axis shows the load (*P*) applied through the load cells at the beam ends, and

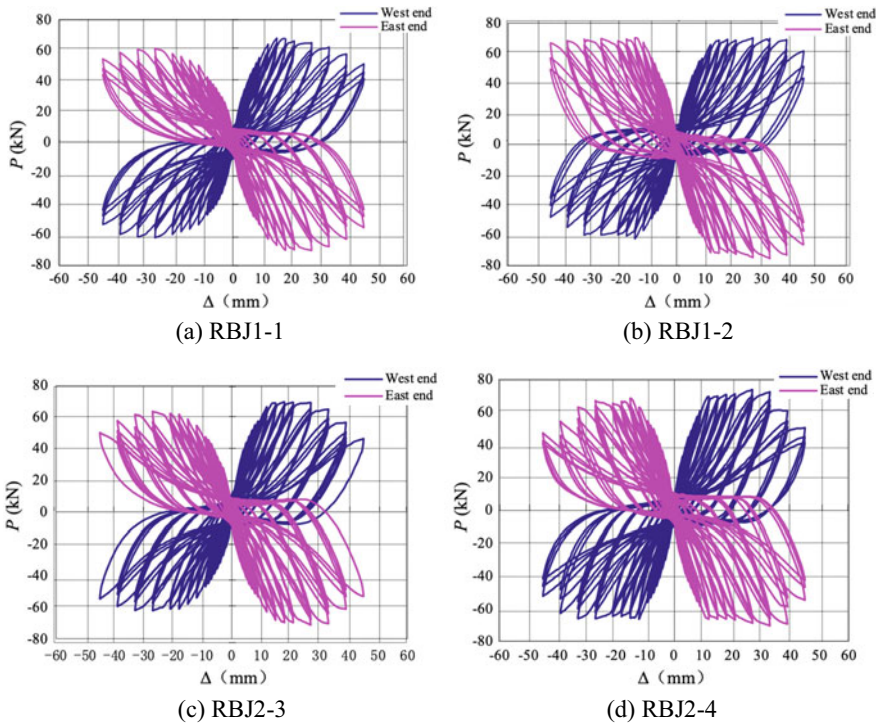


Fig. 5.8 *P-Δ* hysteresis loops of the beam end

the horizontal axis shows the vertical displacements (Δ) of the beam ends. In the figure, for the convenience of presentation, the displacement is considered as positive when the actuators push, and the push force at the west (on the right in Fig. 5.5b) end is considered as positive while the push force at the east (on the left in Fig. 5.5b) end is defined as negative. Since the area enclosed by the hysteresis loop indicates the energy dissipation in each cycle, these plots give a visual representation of the energy dissipation. It can be found from the hysteresis loops that the specimens are basically at the elastic state in the first two cycles and the residual deformation is very small. The beam end displacement increases in the third cycle of loading and the area of hysteresis loop starts to increase, showing a relative increase of energy dissipation capacity. When the specimens entered into the plastic stage, the curves exhibited pinching characteristic. The pinching of the hysteresis loops was caused by the close and open of cracks or debonding of the reinforcements.

After the yielding of steel and the concrete spalling on the beam's upper surface, the stiffness decreased rapidly, as shown in the displacement-controlled loop, and there was no further increase of strength. The majority of the energy was dissipated through the flexural yielding at the plastic hinge generated by the loading at the beam ends. The area of the hysteresis loops gradually increased in each cycle until the displacement limit of the loading devices was reached, with the maximum value up to about 45 mm (about 1/25 beam length) in one direction.

5.4.3 Analysis of Ductility

Skeleton curves are consisted of the maximum peak of each cycle of the hysteresis loops, and Fig. 5.9 shows the skeleton curve of the west beam end. It can be found from the skeleton curves that because of the same reinforcement ratio of the RC beams all the curves are very close to each other, especially at the elastic stage. After the maximum load, the strength can sustain with the growth of deformation.

Ductility and energy-absorption capacity are important considerations in a seismic design. The design strength of a structure can be substantially reduced if the structure is able to provide a good deformation capacity beyond the elastic limit. These quantities are now defined and evaluated for the test specimens. There are many ways to define the ductility of structures. One way is to define the ductility as the ratio of the displacement corresponding to the maximum load (Δ_m) and the nominal yielding displacement (Δ_y), where the yield displacement is determined by the yield moment method (Paulay and Priestley 1992; Wang et al. 2008), as shown in Fig. 5.10.

$$\mu_m = \Delta_m / \Delta_y \quad (5.1)$$

where μ_m is the ductility ratio at the maximum load. Although this definition of ductility is physically clear, the load-deflection characteristics are not fully incorporated in this index. Another definition of ductility is to introduce a collapse point of a member where the load resistance is reduced to 85% of the maximum load (Kuang

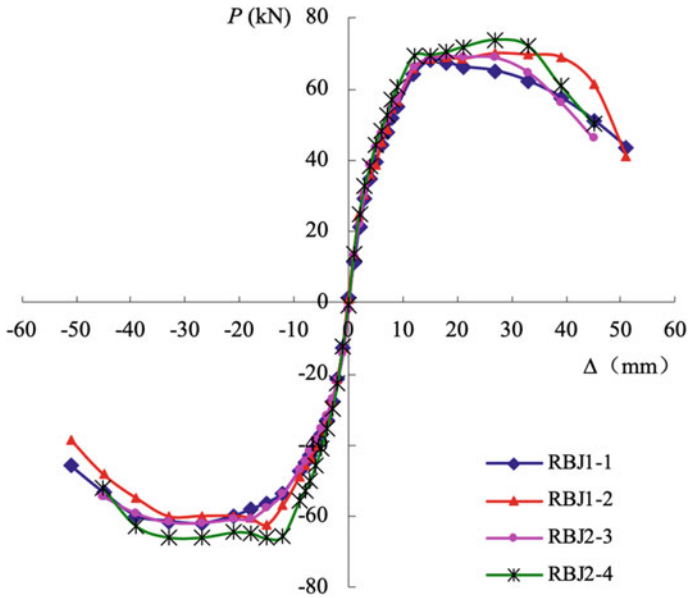
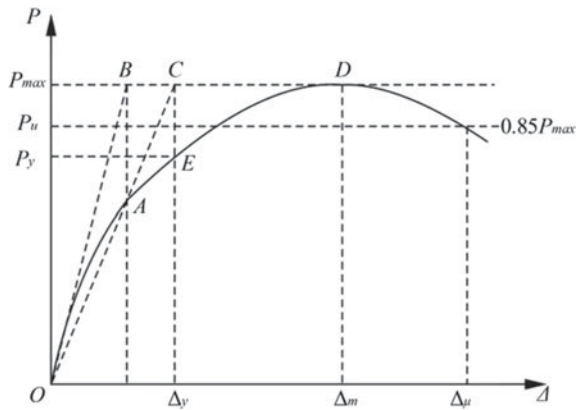


Fig. 5.9 Skeleton curves of west beam ends

Fig. 5.10 Determination for yield point of specimens



and Wong 2005). The defined collapse point may not be the real collapse of a specimen, but is intended to represent a particular damage state of a specimen. Then, the ductility in this model is also defined by

$$\mu_{\mu} = \Delta_{\mu} / \Delta_y \tag{5.2}$$

Table 5.4 Maximum load and ductility index

Specimen number	P_m (kN)	Δ_y (mm)	Δ_m (mm)	Δ_μ (mm)	μ_m	μ_μ
RBJ1-1	65.425	7.9	23.1	46.6	2.93	5.92
RBJ1-2	69.45	6.6	21.9	46.65	2.84	6.12
RBJ2-3	66.375	7.6	28.8	40.75	3.82	5.40
RBJ2-4	71.1	7.7	25.9	40.1	3.39	5.22

where μ_μ is the ductility ratio at the collapse point, and Δ_μ is the collapse displacement. The maximum beam end load (P_m) and the corresponding beam end displacement for each test specimen are summarized in Table 5.4, together with the calculated values of the two ductility factors μ_m and μ_μ . All the values are the mean value of the west and east end including both push and pull directions.

As can be seen from Table 5.4, all the specimens have excellent ductility at the collapse point, and the reinforcement ratio of the ring beam has an obvious influence on the load carrying capacity of the joints by comparing RBJ1-1 to RBJ1-2, and RBJ2-3 to RBJ2-4, respectively. The maximum load of RBJ1-2 and RBJ2-4 has improved by near 10% than RBJ1-1 and RBJ2-3 respectively. With the improvement of the inner tube, the capacity of RBJ2-3 and RBJ2-4 has also improved compared with RBJ1-1 and RBJ1-2, but the ductility at the maximum load increased significantly because of the decrease of the beam-to-column linear stiffness ratio.

5.4.4 Load-Strain Curves

The strains of the transverse reinforcement in the ring beam of RBJ1-1, the one with the smaller inner tube and the lower reinforcement ratio, are shown in Fig. 5.11. It shows there is an initial strain of $100\mu\epsilon$ after the axial force applied at the top of the column, which indicates the confinement of transverse reinforcement to the concrete core (positive value under tension). With the increase in the number of loading cycles, the strain increased and was more than $1300\mu\epsilon$. However, the transverse steel bars did not yet yield, and they offered sufficient transverse confinement even until the structural failure. Because of the contribution of the compressive concrete, the strain of the tensile steel increased more obviously than the compressive steel under the cycling beam end load.

The strains of both inner and outer steel tubes of RBJ1-1 are shown in Figs. 5.12 and 5.13. The strain gauges were located at the both sides 100 mm above the surface of the beam. It shows that after the axial force applied at the top of the column, the average value of the initial longitudinal strain of all the top and bottom gauging points on the inner tube is $482\mu\epsilon$, larger than the outer tube value of $243\mu\epsilon$. The main reason is that there still has some shear-slip phenomenon between the outer tube and concrete. The transverse strain of the outer tube was larger than the inner

Fig. 5.11 Load-strain curves of transverse reinforcement

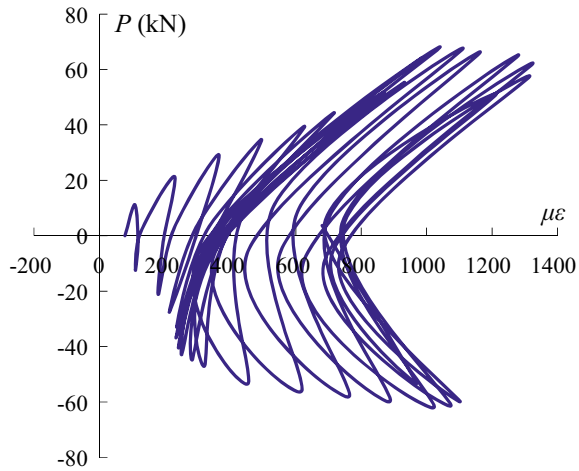
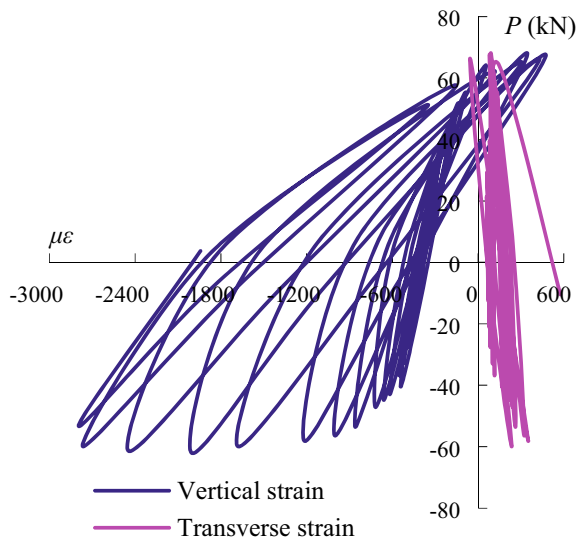


Fig. 5.12 Load-strain curves of inner tube



tube because the outer tube provided more confinement to the concrete while the inner tube was surrounded by the outer concrete in the column. Although the outer tube was discontinuous in the joint core, the transverse and longitudinal strain values were around $1500\mu\epsilon$ at the ultimate state, and it could work well together with the concrete. However, concrete cracking tended to isolate the outer steel tube from the joint core. As in a reinforced connection, the separation can be reduced by the shear connectors and sufficient anchorage of the vertical rebars.

The strains of the vertical rebars of RBJ1-1 are shown in Fig. 5.14 where the strain gauges were located at the middle points of the rebars. It can be seen that because

Fig. 5.13 Load-strain curves of outer tube

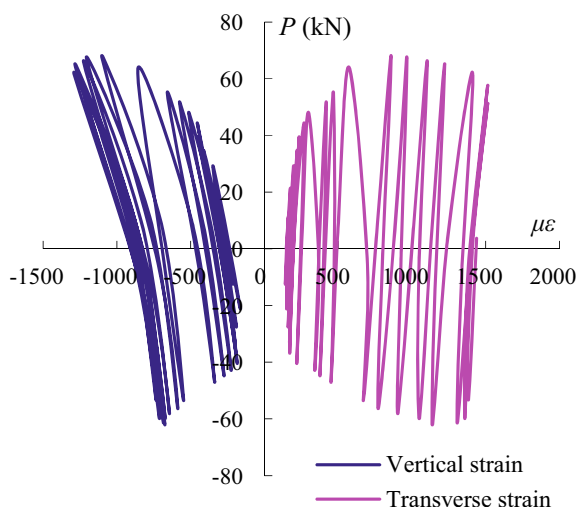
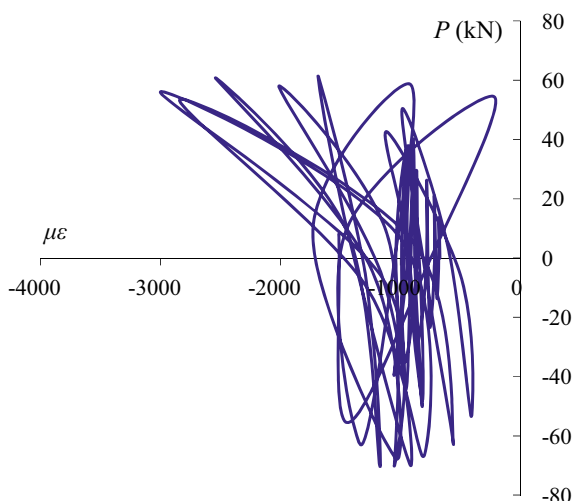


Fig. 5.14 Load-strain curves of vertical reinforcement



the potential eccentricity of the axial compression force applied to the column, some initial strain may be as high as $850\mu\epsilon$ after the axial force was applied, while the average initial strain of all the vertical reinforcements was $554\mu\epsilon$, nearly close to the value of the inner steel tube. Therefore, the vertical reinforcement can provide a resistance of compression instead of the outer steel tube. With the increase in the number of loading cycles, the strain increased and the rebars yielded. The measured rebar stress indicated that the force transfer of the vertical reinforcement in the composite joints can be generally achieved.

5.5 FE Modeling

5.5.1 Comparison between Test and FEM Results

The following sections present a nonlinear FE numerical investigation carried out on the ring beam joint with a discontinuous outer tube to further enhance the understanding of its complex behavior. In recent years, advancements in FE formulation have produced robust algorithms to deal with complex geometry, large deformation, plasticity, and contact interactions (Varma et al. 2002; Tort and Hajjar 2010; Zienkiewicz et al. 2005). However, numerical modeling of this kind of connection is difficult as the model must respond to the nonlinear behavior of reinforced concrete such as concrete cracking, concrete plasticity, concrete crushing, and yielding of reinforcement, in addition to yielding of the steel tubes. Here ANSYS element Solid65 was chosen for concrete, in which the William and Warnke failure criterion was used for both cracking and crushing failure modes through a dispersed modeling approach (Hajjar et al. 1997; William and Warnke 1975). The three-dimensional isotropic 8-node solid element Solid 45 was used to simulate the steel tubes. The Von-Mises criterion and bilinear kinematical hardening stress-strain relationships have been used for steel (Varma et al. 2002). For steel rebars, element Link8 was used and Combin39 was chosen to simulate the bond-slip behavior between the concrete and reinforcement.

The steel tube stress distribution of every analysis step can be obtained through the ANSYS POST26 preprocessor. Figure 5.15 shows the Mises stress distribution of the steel tubes. The maximum stress of the inner tube happened at the cross-section where the outer tube was interrupted because the inner tube was under a complex

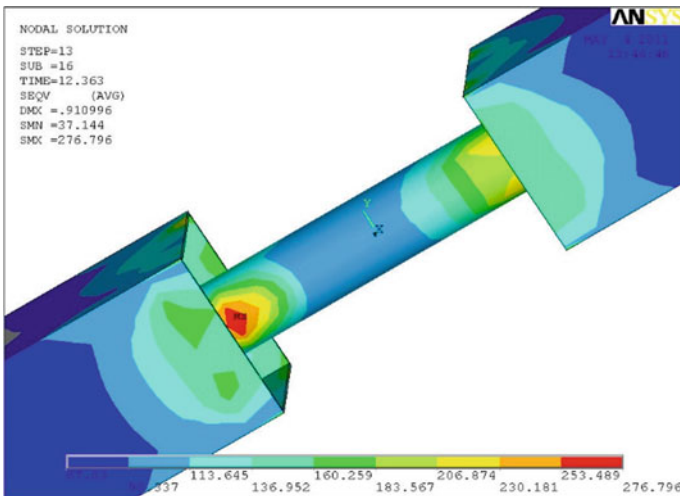


Fig. 5.15 Stress distribution of steel tubes

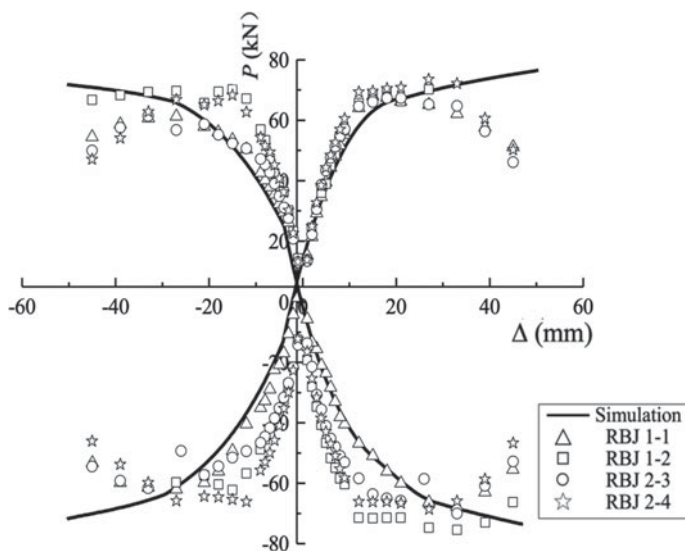


Fig. 5.16 Comparison of load-displacement skeleton curves

stress state with more compression, bending and shearing. When the outer tube was interrupted at the joint zone, there was expected stress concentration at the vicinity of interruption, so it is essential to weld shear connectors on the inner wall of the outer tube to ensure sufficient bond strength.

The comparison of load-displacement skeleton curve between FE simulation and the experimental results is shown in Fig. 5.16. The solid lines stand for the FE simulation results for Specimen RBJ1-1, and the scatter plots stand for the experimental results. It shows the initial stiffness of the FE simulation is lower than that of the actual test results. The ANSYS response deviates a little from the experiment before the peak point obtained from the test. It however predicts almost the ultimate load accurately. Afterwards, there is a certain difference between the experimental and numerical skeleton curves owing to the significant variations of concrete. Therefore, though the loading decline period of the testing specimen was not modeled well, this model can be considered acceptable and is used for a further study.

5.5.2 Parametric Studies

According to the discussions of the FE models, a parametric study was performed to predict the behavior of ring beam joints with a discontinuous outer tube by varying critical parameters. The structural response of the joints was studied by varying some key parameters such as the anchorage length l_a and cross-section ratio P_v of the vertical reinforcement, $P_v = A_v/A_{ot}$, where A_v is the cross-section area of the

vertical reinforcement, and A_{ot} is the cross-section area of the outer steel tube. When the outer steel tube is interrupted in the joint, the cross-section area of the vertical reinforcement should be chosen as same as the outer steel tube theoretically, but it is difficult to realize this in practice for concrete casting.

In the test, $P_v = 0.6$ was chosen for the easy construction of the specimens, and the anchorage length l_a is 400 mm, so test models shown in Fig. 5.16 were simulated with the same parameters as $P_v = 0.6$ and $l_a = 400$ mm. In this FE modeling, the inner tube was taken as $\Phi 140 \times 3.5$ mm² and the volume reinforcement ratio of the ring beam was 0.0015 as same as the test specimen RBJ1-1. Different values of l_a , P_v were considered, then the relationship between l_a , P_v and the skeleton curves are shown in Figs. 5.17 and 5.18, respectively. It can be found from the figure that the bearing capacity can both be improved with the increase of the both parameters. Small change was found in the initial stiffness of the joints before the joints yielded. Although all specimens have the same RC beams, the connection condition can

Fig. 5.17 Influence of l_a on skeleton curves ($P_v = 0.6$)

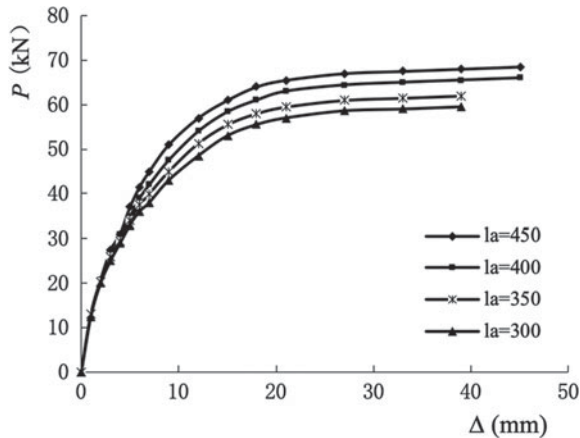
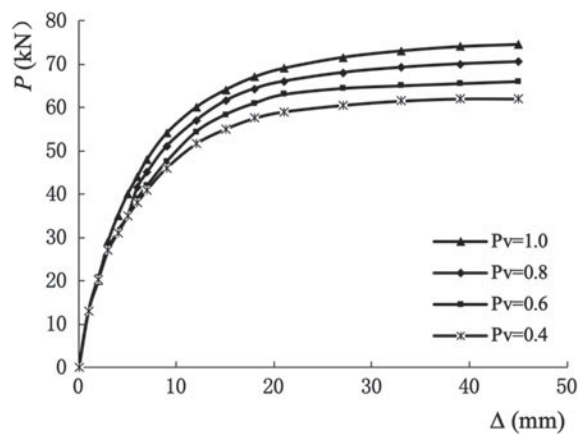


Fig. 5.18 Influence of P_v on skeleton curves ($l_a = 400$)



influence the bearing capacity and ultimate displacement. For example, when $l_a = 300$ mm, the force transfer in joints will not be completely achieved due to the slipping of the vertical rebars in concrete, resulting in a low load bearing capacity. As observed in the experimental work, the FE model clearly demonstrated the benefit of the anchorage of the vertical reinforcement within the concrete in reducing the slipping. Therefore, it is applicable in applications that l_a was taken as 400 mm in the aforementioned test and take $l_a = 400$ mm to study the influence of P_v . As can be seen from Fig. 5.18, the load bearing capacity can be significantly improved with an increase of P_v . The ultimate load was increased approximately by 20% as the cross-section ratio of the vertical rebars was increased from 0.4 to 1.0. Therefore, if the area of the vertical reinforcement was taken the same as the outer steel tube, the ultimate load of test would have been improved by 12%. Furthermore, because the moment was mainly transmitted through the vertical rebars in this connection, more reinforcement with a sufficient anchorage length can achieve a higher load capacity.

5.6 Summary

This part presented a connection system between the RC beam and the CFDST column. Four beam-to-column assemblage specimens subjected to cyclic loads were tested, and seismic behaviors of the joints including the load-deflection performance, typical failure modes, stress and strain distributions, and energy dissipation capacity were studied. Finite element modeling was also implemented to conduct some parametric analyses. Based on these investigations from both experimental and numerical approaches, the conclusions are summarized as follows.

- (1) The connections belong to the through-beam connection family in which the outer steel tube is interrupted to make the longitudinal reinforcement of RC beams continuous in the joint core. The beam moment and shear force can easily be transmitted by the RC beam. Details such as the vertical reinforcements, welded shear studs, and the ring beam ensured integrity of the joint to achieve an ideal rigid connection. Meanwhile, the continuous inner CFST played an important role in keeping rigidity and integrity of the joint, especially improving the shear resistance.
- (2) The plastic hinge is predicted to develop at the junction between the ring beam and RC beam. The yielding of the beam was followed by the propagation of cracks at the beam end, while the joints and columns maintained a stable status. The ring beam joint with a discontinuous outer tube, when designed with suitable parameters, performed quite well in carrying the cyclic loads as they could generally attain their strength and deformation capacity. The joint exhibits good seismic resistance until the beam is totally damaged and the “strong joints” could be achieved. This connection system provides adequate stiffness and strength for a “weak beam & strong column” system, and therefore offers a viable connection for seismic resistance design.

- (3) The reinforcement ratio of the ring beam showed an obvious influence on load-carrying capacity of the joints, and the ductility at the maximum load was improved with the strengthening of the inner tube. The numerical study clearly suggests that anchorage length l_a and cross-section ratio P_v of the vertical reinforcement are critical parameters in the behavior analysis of ring beam joints with a discontinuous outer tube.

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Chapter 6

Vertical Stiffener Joints to Steel Beams



Abstract Miao (2004) and Chen and Miao (2005) studied the vertical stiffener joints for the square CFST column respectively. Those analysis results showed that vertical stiffener joints with simple and reasonable construction details had the advantage of transferring load reliably. As the CFDST column has double steel tubes, the vertical stiffener connection between CFDST and H-shaped steel beam was designed in 2012 (Zhang et al. 2015, 2018). The proposed connection mainly consists of vertical stiffeners and embedded anchorage webs, which was devised to cater for the seismic requirements of “strong column and weak beam”. This design was also chosen to provide a relatively strong panel zone so that limited inelastic deformations could occur before significant flexural yielding developed in beams. The structure form, mechanical characteristics and force transfer mechanism of this new type of joint were analyzed and summarized. The behavior of the new joints under seismic loads was studied through testing six beam-to-column assemblage specimens, and for comparison one specimen was conducted between a square CFST column and steel beams.

6.1 Structure Forms and Features of the Connection

The joint specimens are intended to simulate the connection between an interior column and two adjacent steel beams in a frame structure. The length L and height h of the test specimen are taken on the base of the assumption that the points of inflection are at the midspan and mid-height of the prototype beams and columns, respectively. The key of this connection system is that vertical stiffeners are served as the primary force transfer member. The vertical stiffener is groove welded on the column flange, and its overhang is connected to the endplate, so the overhang is as long as the widened endplate. To avoid the stress concentration, the curved section is used for the endplate. The endplate as wide as the column flange is butt-welded to the beam flange. It can be realized that the widened endplate induces the plastic hinge on the beam in order to prevent brittle fracture of the columns. The anchorage web welded on the inner circular tube connects with the beam web using connecting plates and bolts. The application of anchorage web embedded in the

concrete between the inner tube and outer tube can improve workability and integrity of the joint. Because the anchorage web embedded inside the concrete is higher than the beam web, and the anchorage web can be chosen as two types including with triangle ribs and without ribs. In the ribbed connection, the outer tube is slotted to realize double-bevel groove welds to the ribbed anchorage web. Therefore, this fully restrained connection is commonly characterized by: (1) vertical stiffener; (2) anchorage web; and (3) reduced beam section.

6.2 Test Specimens and Material Properties

The experimental study was used to comprehend the force transfer mechanisms between the CFDST columns and steel beams. More consideration focused on the connection's failure mode, ductility, strength, energy dissipation and hysteretic behavior for seismic resistance. Six joint specimens between CFDST columns and steel beams, and one contrast connection between the square CFST column and the steel beam were designed on a 1:2 scale, named by SBJ1-1, SBJ1-2, SBJ2-1, SBJ2-2, SBJ3-1, SBJ3-2 and SBJ4-1 respectively. Table 6.1 lists the key features of these seven specimens tested in this investigation. The level of friction-type high-strength bolt M20 is 10.9. The bolts were tightened with a special torque wrench according to design requirements. The variables in the tested specimens include overhang of the vertical stiffener (the same length as the widened endplate), anchorage web with or without ribs, width-thickness ratio of the square steel tube, and axial compression

Table 6.1 Parameters of specimens

Specimen number	Outer tube (side length $\times$ thickness) $\square B \times t$	Inner tube (diameter $\times$ thickness) $\Phi 2r_i \times t_i$	Overhang of vertical stiffener (mm)	Axial compression ratio	Anchorage web
SBJ1-1	$\square 250\text{mm} \times 8\text{mm}$	$\Phi 133\text{mm} \times 6\text{mm}$	80	0.23	Without ribs
SBJ1-2	$\square 250\text{mm} \times 8\text{mm}$	$\Phi 133\text{mm} \times 6\text{mm}$	120	0.23	Without ribs
SBJ2-1	$\square 250\text{mm} \times 8\text{mm}$	$\Phi 133\text{mm} \times 6\text{mm}$	80	0.23	With ribs
SBJ2-2	$\square 250\text{mm} \times 8\text{mm}$	$\Phi 133\text{mm} \times 6\text{mm}$	120	0.23	With ribs
SBJ3-1	$\square 250\text{mm} \times 8\text{mm}$	$\Phi 133\text{mm} \times 6\text{mm}$	80	0.40	Without ribs
SBJ3-2	$\square 250\text{mm} \times 5\text{mm}$	$\Phi 133\text{mm} \times 6\text{mm}$	80	0.23	With ribs
SBJ4-1	$\square 250\text{mm} \times 5\text{mm}$	—	80	0.23	Through

ratio. The axial compression ratio is a ratio of axial force N at the column top to calculated ultimate bearing capacity N_u using Unified Theory of CFST, which can refer to Chap. 2. The main dimensions and shapes of the specimen are shown in Fig. 6.1, where $L = 2450$ mm and $h = 1944$ mm. The H-shaped steel beam section is $H \times B \times t_1 \times t_2$: $244 \times 175 \times 7 \times 11$ mm⁴.

The connection ichnography details for the CFDST column are shown in Fig. 6.2, where the overhang of the vertical stiffener is 80 mm long. Elevation views of CFDST column-to-beam joints including the unribbed joint SBJ1-1 and the ribbed joint SBJ2-1 are shown in Fig. 6.3. Complete-penetration single-bevel groove welds were used to connect the endplates to the column flange in all specimens. All continuous plates were welded to columns using partial-penetration double-bevel groove welds. The connection details for the square CFST column are shown in Fig. 6.4, where the anchorage web without ribs is through the joint core. The photograph of the specimens is shown in Fig. 6.5.

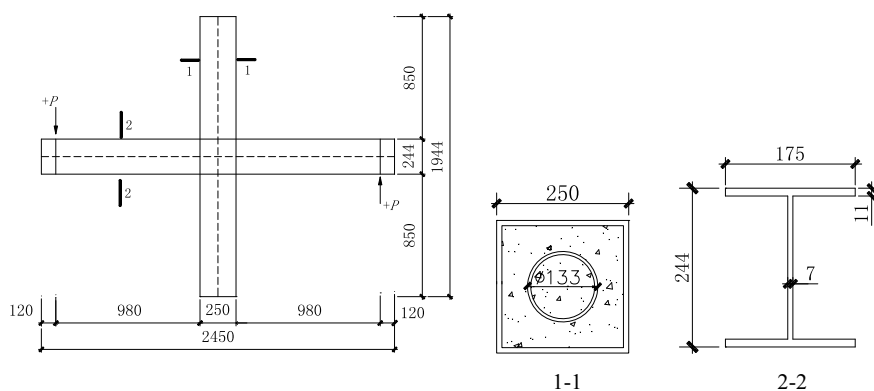


Fig. 6.1 Main dimensions of the specimen

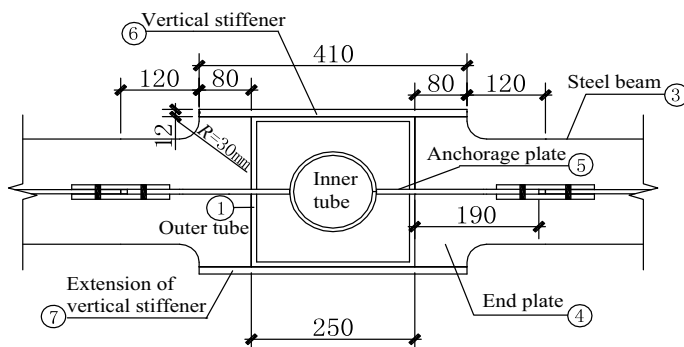


Fig. 6.2 Ichnography details of the CFDST column-to-beam joint

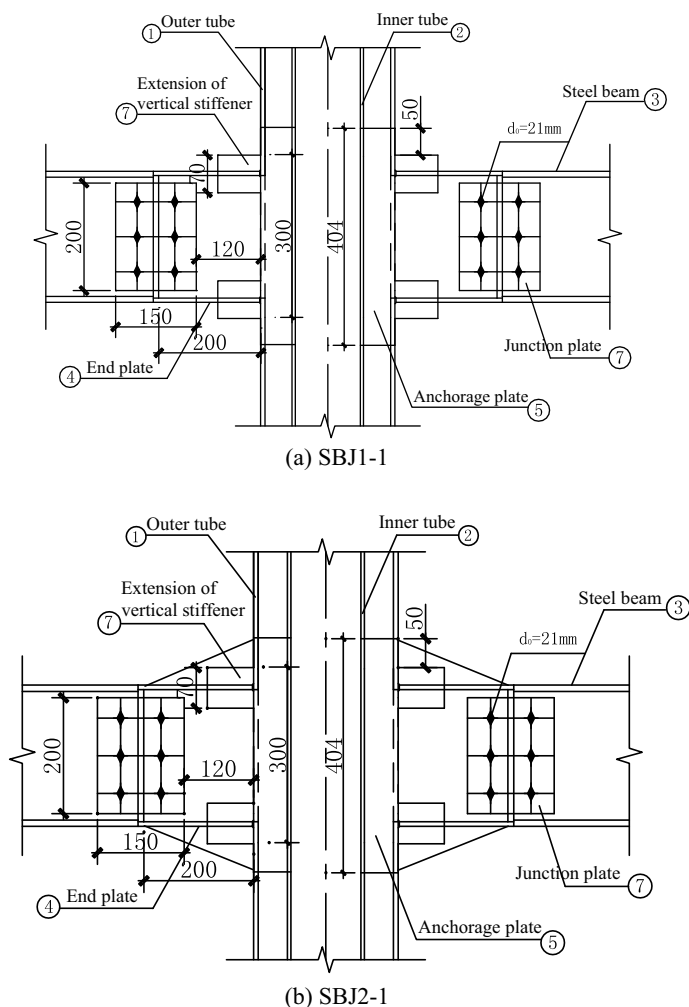


Fig. 6.3 Elevation view of the CFDST column-to-beam joint

The steel strength and elastic modulus were measured according to the Chinese standard GB/T 228-2002. Expansion agent was added into the concrete to ensure bonding performance between concrete and steel tubes. The compressive strength of the concrete was measured by using concrete cubes with side length of 150 mm. It got $f_{cu} = 59.5$ MPa. The material properties of the steel plates and tubes for the beams and columns are given in Table 6.2. The material properties listed in Table 6.2 were obtained through standard material test methods. The steel of anchorage web is the same steel as the square tube, so is the vertical stiffener.

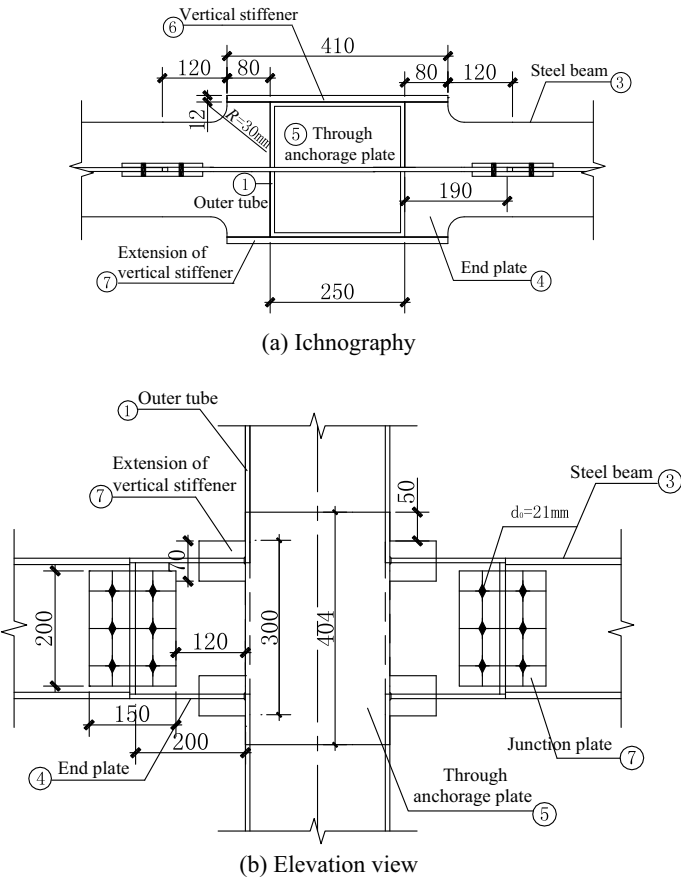


Fig. 6.4 Details of the square CFST column-to-beam joint (SBJ4-1)

Fig. 6.5 Fabricated specimens



Table 6.2 Material properties of the steel

Steel material	Square tube	Circle tube	Endplate	Beam flange	Beam web
Average thickness (mm)	7.6	6.4	11.5	10.6	6.6
Yield strength (MPa)	338.12	323.08	272.61	272.41	291.00
Ultimate strength (MPa)	481.70	491.39	445.86	447.40	457.23
Elastic modulus (10 ⁵ MPa)	2.257	2.152	2.132	2.213	2.157

6.3 Test Setup and Measurements

The purpose of the experimental study was to investigate the mechanical properties of the tested joints under earthquake action. As noted in Chap. 5, the same test setup and self-equilibrating reaction frame were adopted in this test. As shown in Fig. 5.5a, a 2000 kN vertical hydraulic jack was placed at the top of the column and used to apply a constant vertical force to simulate the effect of the upper floors in a multi-storey building, and a spherical hinge was placed between the actuator and the column to simulate the inflection point of the joint specimen. The column end was restrained by a rigid transverse beam connected to the reaction wall, but allowed to freely rotate in the loading plane. A vertical low cyclic reciprocating load was applied to each end of the beam by an MTS hydraulic actuator to simulate the seismic loading effect. The test photos are shown in Fig. 6.6. Four specially designed out-of-plane lateral braces were used to avoid torsion of the joint specimen during the loading process, as shown in Fig. 6.7. According to the suggestion of ATC-24 (1992), a displacement control with three cycles at each displacement amplitude was used after the specimen yielded. The displacement increments between each cycle were 3 mm one time before yielding. Then, the amplitude increment of vertical displacement was changed to ± 6 mm and cycled three times. The loading process

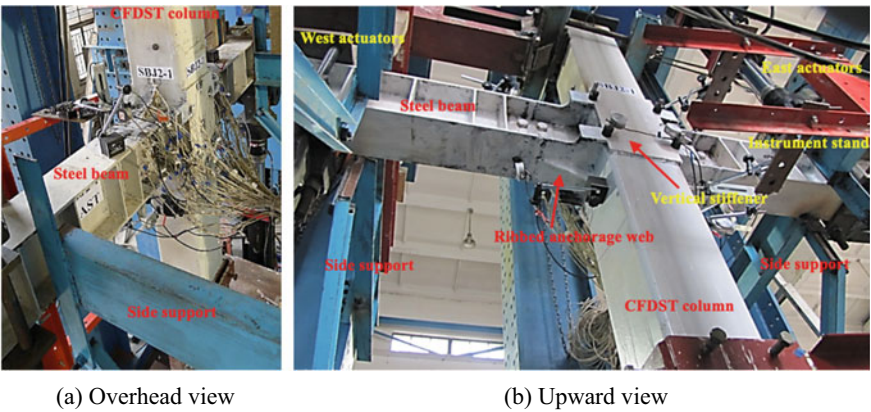


Fig. 6.6 Photographs of specimen test

Fig. 6.7 Lateral braces of steel beam



was terminated until failure of the specimen or the load below 80% of the maximum. All tests were conducted by servo control hydraulic actuators, with a preprogrammed displacement history.

Strain gauges were glued on the inner steel tube in advance to measure vertical and transverse strains, and the other strain gauges for steel beams and outer steel tubes were glued on the steel surface before testing. Figure 6.8 shows the arrangement of strain gauges, together with displacement transducers. The strains of the steel web were measured at three different locations by means of rosette strain gauges in order to determine the distribution of stresses.

6.4 Discussion of Test Results

6.4.1 Failure Modes

The specimens with the weak beam configuration were designed to make yielding develop primarily in the beams to investigate the performance of the panel zone in the weak-beam-strong-column system. In general, the plastic hinge formed as expected in the reduced beam section where the endplate was cut as curved. Firstly, the CF DST column was compressed and the beam ends went down freely under the designed axial load. Then, during the cyclic loads at beam ends, the failure process of each specimen can basically be divided into three stages: the elastic stage, the deformation development stage and the failure stage. At the elastic stage, load was roughly linear related with the displacement of the beam end, and shear deformation of the panel zone was very small. Afterwards, with the load increasing, the displacement developed rapidly. And the relationship between load and displacement of the beam end

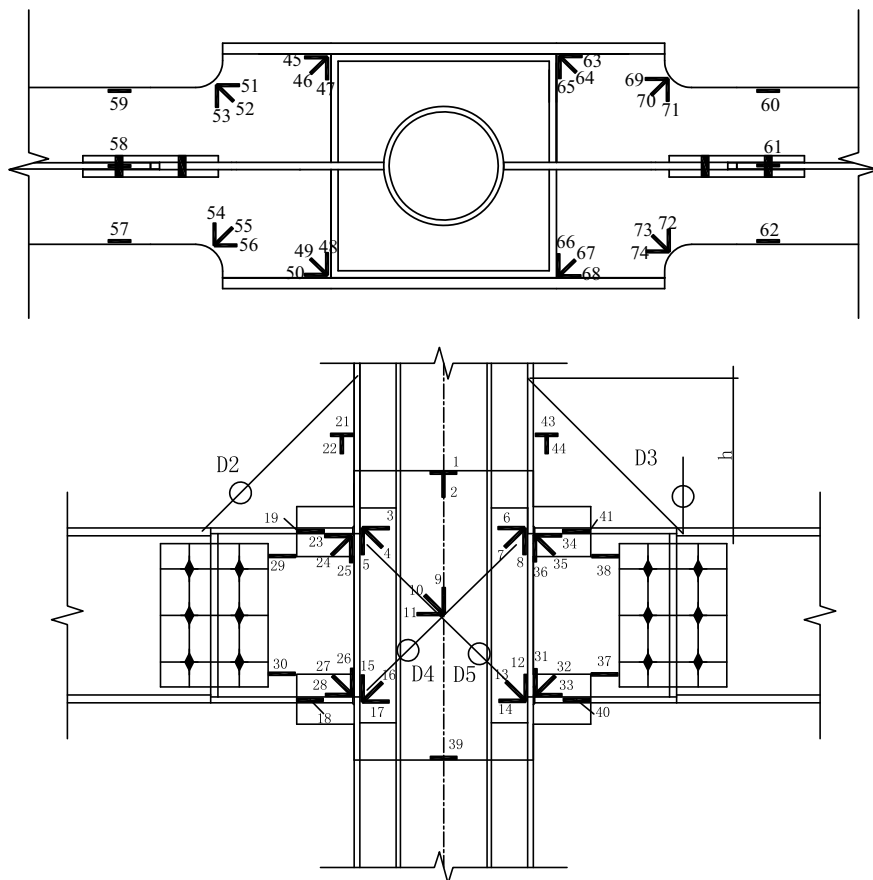


Fig. 6.8 Arrangement of strain gauges and deformation transducers

was no longer linear. According to the strain measurement, steel at the curvature of the horizontal endplate yielded firstly, and then the steel at the junction between the beam flange and column yielded. The shear deformation of the panel zone increased but was still in an elastic state.

The failure patterns of all specimens for the new connection system tested herein are shown in Fig. 6.9. The failure modes of each specimen are listed in Table 6.3, in responding to the failure pictures. Generally, two distinct cracks initiated at the horizontal endplate, as shown in Fig. 6.9a. These cracks then propagated along the thickness and width of the widened endplate. On the other hand, for Specimens SBJ1-2 and 2-1, the crack propagated almost through the entire beam flange, as shown in Fig. 6.9c, f. Specimens SBJ1-1, 2-1 and 3-1 with the 80 mm overhang of vertical stiffener failed starting from the break at the curvature of the horizontal endplate, and some beam-to-column junctions fractured, as shown in Fig. 6.9b; while Specimens SBJ1-2 and 2-2 with the 120 mm overhang failed starting from the steel beam flange,



Fig. 6.9 Typical failure patterns of the specimens

Table 6.3 Summary of failure modes

Specimen numbers	East beam		West beam	
	Top flange	Bottom flange	Top flange	Bottom flange
SBJ1-1	a	a, b	a, b	b, d
SBJ1-2	c, g	c	d	—
SBJ2-1	f, g	f, h	a, b	b, d
SBJ2-2	f, g	f	f, g	e
SBJ3-1	a, b	a	b, d	a, b
SBJ3-2	—	—	—	—
SBJ4-1	—	b	b	—

Annotation: ‘—’ means fracture at weld seam

and the beam-to-column junction was intact. Therefore, the plastic hinge is removed away from the surface of the column when the overhang of the vertical stiffeners is longer. For example, in the case of the ribbed specimen SBJ2-1 with the 80 mm overhang, strong stress concentration on the place of ribs resulted in tearing of the steel wall, as shown in Fig. 6.9h, while the phenomenon of tearing didn't happen to the ribbed specimen SBJ2-2 with the 120 mm overhang. It can be concluded that the beam-to-column junction was protected well, then significant yielding along the beam flange is observed, as shown in Fig. 6.9e.

As shown in Table 6.3, some fractures at weld seam happened by accident, but it is also unavoidable because the bending moment was ultimately transmitted through the attachment weld between the overhang of vertical stiffener and endplate, outer steel tube and endplate respectively. The attachment weld is the important place likely to be failed first, which should be avoided in practice. As welding generally deteriorates material toughness in the heat-affected zone and the weld itself is prone to defects. SBJ3-2 was the first specimen to be conducted in the tests. The premature fracture has occurred in the weld seam shortly after the yield load. Cracks initiated at the beam-to-column junction and partial tear of the column flange occurred, then attachment weld fractured accompanied by sound. It appears that the quality of the flange groove welds, adjacent to the beginning and endpoints of each pass of these welds, may need to be viewed with suspicion. So, weld reinforcement has been done for the following joint specimens, and the better failure mode was obtained eventually. In addition, for Specimen SBJ3-2 with a thinner steel tube, the beam-to-column junction and the column flange were torn earlier, so the thicker outer steel tube can ensure a good plane tensile property. Meanwhile, increasing the local thickness of the steel tube may be worth considering in the practical design because it is expensive to increase the overall thickness of the steel tube.

The measured strain values were very small in the joint panel zone (basically under the half value of the steel yield strain), so specimens designed with "strong" panel zones make deformation concentrate on the beams, and the panel zone is remaining essentially elastic. From the observation, it is confirmed that the destruction of the joint is that plastic hinge forms at the steel beam.

6.4.2 *P-Δ Hysteresis Loops*

The hysteresis loops represent the relationship between the applied displacement and the corresponding load recorded during the cyclic loading tests. The horizontal axis shows the vertical displacements of the beam ends, and the vertical axis shows the corresponding applied force through the load cells at the beam ends. In the figures, for the convenience of presentation, the force is considered as positive when the actuators push, and negative when the actuators pull. However, it is different for the expression of displacement sign. The pushing displacement of the west end is considered as positive while the pushing displacement of the east end is defined as

negative. Upon that, responses of each specimen are expressed through the load-displacement hysteresis loops as shown in Fig. 6.10. It was found that except the first cycle following an increase in displacement amplitude, the hysteretic energy per

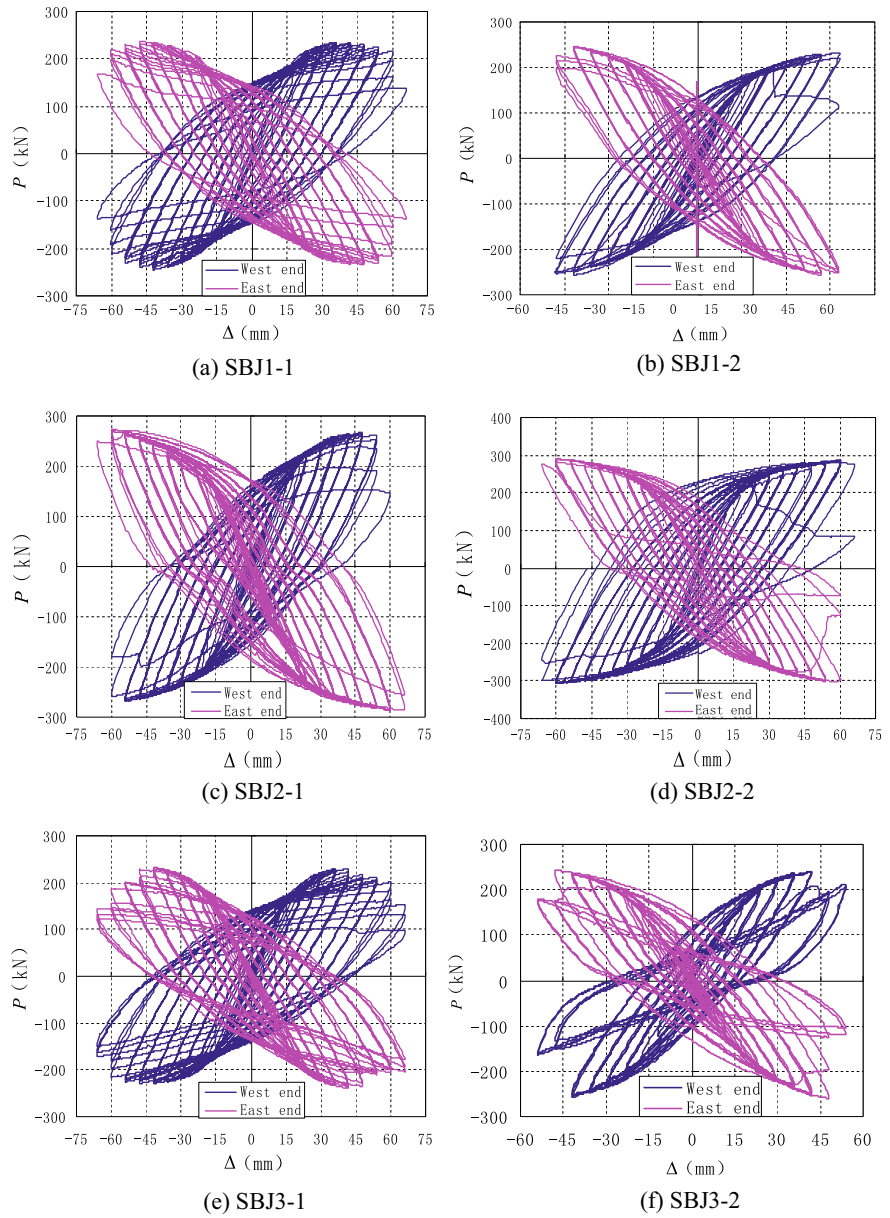


Fig. 6.10 P - Δ hysteresis loops of the beam ends

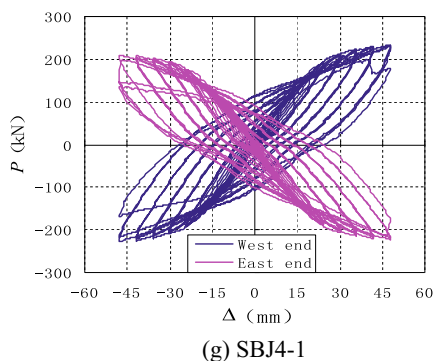


Fig. 6.10 (continued)

cycle (i.e., hysteresis loop area) remained reasonably constant at each displacement amplitude. Further, the ductile behavior of the connections was demonstrated by the increase in hysteresis loop area with each successive increase in displacement amplitude, even with evident pinching at the larger amplitudes. The brittle failure occurred at welds for Specimen SBJ3-2, so its curves exhibited pinching characteristic. In the case of the specimen of square CFST column, SBJ4-1 also showed a little pinching effect in the hysteresis loop. The other curves are relatively full, and the typical hysteresis loop has the following characteristics: the initial stiffness is high and the hysteresis loop straightly goes up, and the residual deformation is very small. When the beam end displacement reaches nearly 24 mm, local buckling and eventual plastic hinge appear at the curvature of the endplate on the steel beam. With the increase of beam end load, residual deformation increases gradually, but the unloading stiffness is the same as the initial loading stiffness. In the elastic-plastic stage, loading stiffness decreases gradually because the yield range of steel beam increases with the increasing deformation. When the specimens enter into the plastic stage, the bearing capacity during the later period is good, and strength degradation is not obvious at the same level-load. Therefore, the test subassemblies exhibit excellent strength and stiffness retention capacity before the fracture failure.

6.4.3 Parametric Analysis

The skeleton curve can reflect the structural strength and deformation properties. Several groups of skeleton curves concerning different parameters are compared in Fig. 6.11. It can be seen from all figures that specimens almost experienced elasticity, yield, maximum load and ultimate stage under the vertical constant load and low levels of cyclic loads. It can be seen from Fig. 6.11a, b that Specimens SBJ2-1 and SBJ2-2 with ribbed anchorage webs compared with SBJ1-1 and SBJ1-2 respectively, initial stiffness and maximum bearing capacity have improved significantly, 17%

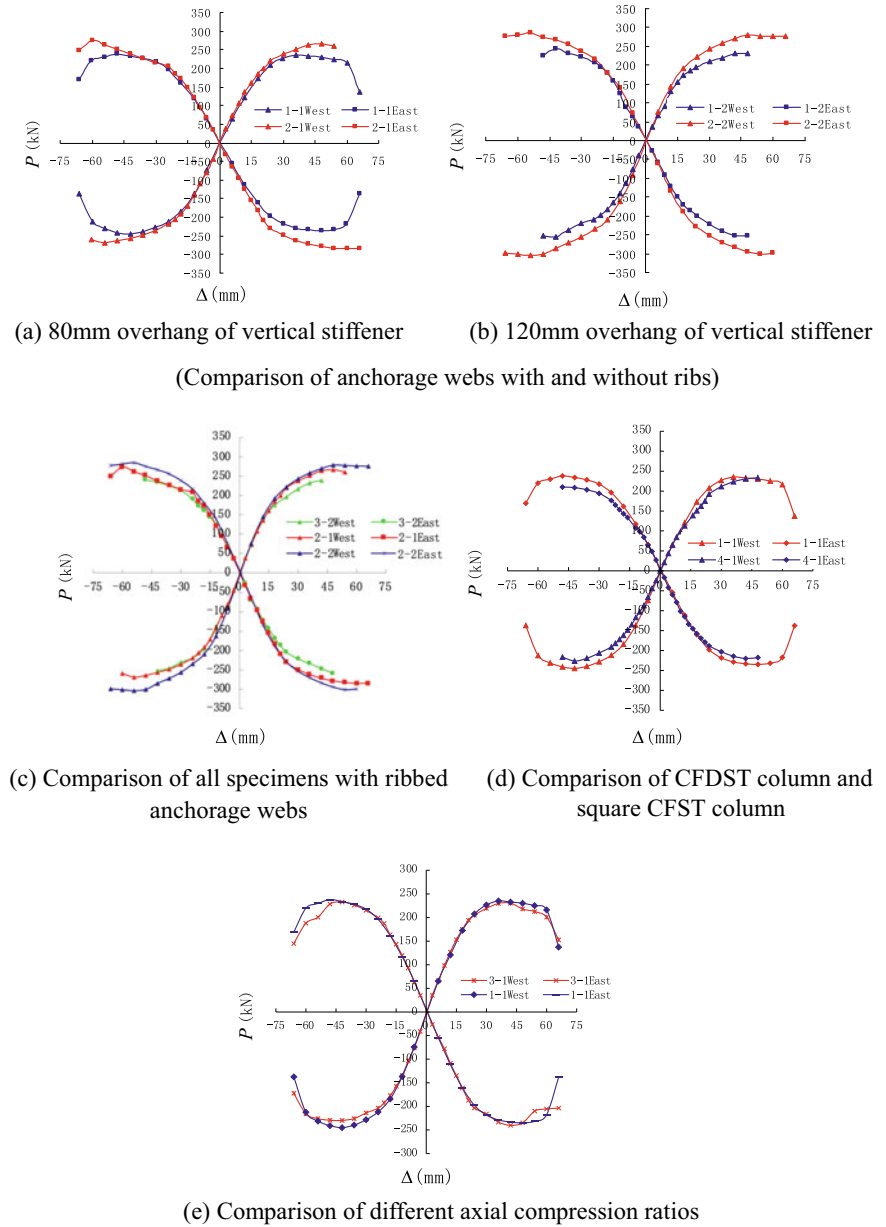


Fig. 6.11 Skeleton curves of beam ends

improved for the maximum bearing capacity. The comparable improvements in load bearing capacity and flexural failure prove that the application of ribbed anchorage webs can improve the joint stiffness and strength. It can be seen from Fig. 6.11c that the strength has developed and sustained with the increase of deformation after the maximum load especially for the ribbed connections SBJ2-1 and 2-2. Specimen SBJ2-2 with the 120 mm overhang of vertical stiffener improves the initial stiffness significantly than Specimen SBJ2-1 with the 80 mm overhang, while the bearing capacity has not been improved much. The main significant difference between the two specimens exists in the failure mode because the plastic hinge can be removed farther from the surface of the column when the overhang is longer. Therefore, the overhang of the vertical stiffener can effectively protect the panel zone of the joint. In the case of Specimens SBJ3-2 and SBJ2-1 with the same overhang, stiffness in the elastic stage has increased due to the increasing beam-to-column linear stiffness ratio. It can be seen from Fig. 6.11d that the initial stiffness of Specimen SBJ1-1 is similar to SBJ4-1 though they have different joint construction. Comparing with the ichnography in Figs. 6.2 and 6.4, the anchorage web of Specimen SBJ4-1 was a whole through the column section at the junction, while for joints of CFDST columns, the anchorage web was interrupted by the inner tube. So, in the case of the CFDST column, the anchorage web can also transfer pulling force by the inner tube to make the joint core a whole, i.e., the anchorage web in the CFDST column is equivalent to the continuous anchorage web. It can be seen from Fig. 6.11e that the strength, stiffness and deformation properties have no significant relation with the axial compression ratio for the CFDST column-to-beam joint.

Based on the foregoing discussions, it can be concluded that the CFDST column-to-beam joints showed sufficient moment resisting capacity than the square CFST column-to-beam joint. In the case of this new type of CFDST column-to-beam joint, it can be found that lengthening the overhang of vertical stiffener will increase shear resistance of the connection to protect the panel zone. The maximum bearing capacity of the connections depends on whether the anchorage web is ribbed or not, so undoubtedly the best way is to use ribbed anchorage webs. The comparison among the composite specimens with different tube thickness suggests that using a thicker tube or increasing local thickness of the steel tube can provide some benefits to the joint performance.

6.4.4 Analysis of Ductility

Ductility and energy-dissipation capacity are important considerations in a seismic design. These properties are now defined and evaluated for the test specimens. Ductility, which can be calculated from the load-displacement curve, is defined as the ratio of ultimate to yielding displacement. This definition of ductility can refer to the definition in Sect. 5.5.3, which is to use a collapse point of a member where the

beam end resistance is reduced to 85% of the maximum load. $\mu_\mu = \Delta_\mu / \Delta_y$, where μ_μ is the ductility ratio at the collapse point, Δ_μ is the collapse displacement, and Δ_y is the nominal yielding displacement determined by the yield moment method (see Fig. 5.10).

The experimental results are also summarized and presented in Table 6.4, together with calculated values of the ductility coefficient. In this test, Specimen SBJ3-2 and three other beam ends have appeared damage before the ultimate state was reached because of the brittle fracture at welds, so the maximum beam end load (P_m) and the corresponding displacement (Δ_m) in the table are blank. As can be seen from Table 6.4, the even value of ductility coefficient is 2.84. Note that the ductility is not actual for the specimens whose bearing capacities suddenly drop without allowing any curvature increment after peak. In structures, the formation of plastic hinges is one of the main sources of inelastic and ductile behavior. The ideal damage is that the steel beam yielded as the formation of plastic hinges and the steel should not be failed due to stress concentration. However, it is difficult to make the structure or components to achieve the desired destruction in practical engineering applications. Therefore, as this chapter stated before, it is very necessary to ensure weld quality, and minimize the welding heat zone. It is helpful to improve the deformation capacity by extending the vertical stiffener or increasing thickness of the outer steel tube.

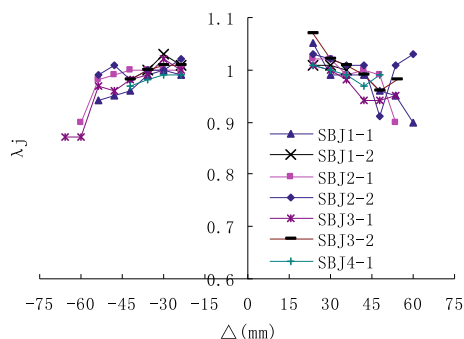
6.4.5 Degradation

A bearing capacity reduction coefficient of the specimens can be written as $\lambda_j = P_j^i / P_j^1$, in which, P_j^i is the peak load of the i th cycle when the displacement control is j , P_j^1 is the peak load of the first cycle when the displacement control is j (Han and Li 2010). After specimens yielded, each displacement level was repeated 3 times. Figure 6.12 shows the changes of bearing capacity reduction coefficient of the west beam end after yielding. It can be seen from the figure that the load value of the later loading cycle compared with the first cyclic loading value almost does not reduce at the same displacement level before the destruction. The bearing capacity reduction coefficient of joint specimen of the CFDST column is almost above 0.94, however, the load will suddenly drop due to serious local buckling deformations such as the west end of Specimen SBJ1-2. Specimen SBJ2-2 has ribbed anchorage web and longer vertical stiffener overhang, and the bearing capacity reduction coefficient is more stable. Therefore, the joint specimen of the CFDST column has stable bearing capacity under low reverse cyclic loading, it can be concluded that this type of connection has a good working performance under seismic action.

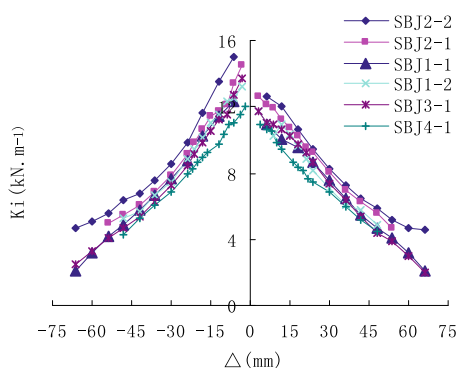
For various reasons, all the specimens suffered the stiffness degradation with the damage accumulation during cyclic loading. The stiffness is estimated by computing slope of the line joining the peak load and zero load at half cycle of each displacement amplitude. The average stiffness at each displacement amplitude can reflect stiffness characteristics with the increase of the displacement. The index of curve stiffness (K_i)

Table 6.4 Test results

Specimens	Direction	P_y (kN)		Δ_y (mm)		P_m (kN)		Δ_m (mm)		P_u (kN)		Δu (mm)		μ	
		West	East	West	East	West	East	West	East	West	East	West	East	West	East
SBJ 1-1	Push	172.8	170.8	18.0	19.6	235.3	238	36	48	200	202.3	62	64	3.44	3.27
	Pull	183.1	171.8	17.8	20.1	245	234.7	42	48	208.3	199.5	62	64	3.48	3.18
SBJ 1-2	Push	172.3	200.6	17.7	23.3	—	243.5	—	42	232	225.5	48	48	2.71	2.06
	Pull	195.9	199.5	21.1	23.5	257	252.9	42	42	251.3	252	48	48	2.27	2.04
SBJ 2-1	Push	192.5	177.1	19.3	19.3	266.7	273.6	52	60	260.5	248.8	54	66	2.80	3.42
	Pull	178.1	188.1	16.7	19.6	268.9	285.1	58	60	259	284.3	60	66	3.59	3.37
SBJ 2-2	Push	202.3	208.4	20.3	22.9	278.7	284.8	48	54	275.9	277.5	66	66	3.25	2.87
	Pull	220.8	195.2	20.7	19.5	303.9	301.5	54	54	299.2	299.5	66	60	3.19	3.08
SBJ 3-1	Push	174.4	194.8	17.7	22.8	230.5	233	42	44	195.9	198.1	58	55	3.27	2.41
	Pull	176.9	196.0	17.9	22.5	229.8	240.1	50	42	195.5	203.7	63	66	3.52	2.92
SBJ 3-2	Push	174.3	180.9	17.9	22.2	—	—	—	—	238.9	240.1	42	48	2.34	2.16
	Pull	197.7	188.4	18.3	21.1	—	—	—	—	251.9	259.7	42	48	2.29	2.27
SBJ 4-1	Push	169.2	136.4	21.2	16.6	—	—	—	—	233.4	210	48	48	2.26	2.88
	Pull	170.2	150.5	20.1	17.2	226.3	220	42	42	216.6	217.8	48	48	2.39	2.79

Fig. 6.12 Curves of bearing capacity degradation

is used to describe the stiffness degradation, which is defined by Nie et al. (2008). Take the average stiffness under same displacement level for: $K_i = \sum_{j=1}^n P_j^i / \sum_{j=1}^n \Delta_j^i$, in which, K_i is the loop stiffness; P_j^i is the peak load value of the i th cycle when the displacement control is j ; Δ_j^i is the corresponding beam end displacement of the i th cycle when the displacement control is j ; n is the load cycle. The relationship between the curve stiffness and the applied displacement of west beam end is shown in Fig. 6.13. It can be seen that the stiffness degradations of all joint specimens are analogous. Stiffness degradation is obvious, continuous, uniform and stable during the entire loading process. The push and pull curves are basically symmetry in positive and negative direction. The initial curve stiffness of push force is lower than pull force because all beam ends were firstly pushed then pulled at each displacement amplitude. Specimen SBJ2-2 with ribbed anchorage webs and longer vertical stiffener overhang has showed higher curve stiffness. Similar to the illustration of bearing capacity, the curve stiffness of joint specimens of the CFDST column is all better than the joint specimen SBJ4-1 of the square CFST column, showing good mechanical behaviors.

Fig. 6.13 Curves of rigidity degradation

6.4.6 Energy Dissipation

Energy dissipation indicates the capability of structures to dissipate energy through a yielding mechanism with satisfactory performance in the inelastic range. After the formation of plastic hinges, deformation of the specimen mainly depends on plastic rotation of the plastic hinge zone. Each of the tests culminated with the formation and subsequent propagation of fatigue cracks. Destruction of the specimens started from the plastic hinge formation at beam end. The hysteresis loops, in spindle form, show good energy dissipation capacity of this type of joint. The energy dissipation capacity can be obtained by integrating the area enclosed by the load-displacement curve and is given by $W = \int P dx$, where P represents load and x represents applied displacement. With increasing of displacement, the energy dissipation increased obviously for all the joint specimens in the post yield stage. The energy dissipation ability can also be evaluated by the equivalent damping coefficient h_e , which is defined as $h_e = E/2\pi$ (Nie et al. 2008). The relationship between equivalent viscous damping coefficient and half-cycles of each specimen is shown in Fig. 6.14. It could be seen that h_e of the latter two cycles was slightly lower than the first cycle at the same displacement level. This is due to energy dissipation generated by slippage and hysteretic response. The equivalent damping viscous coefficients for all specimens increased with the increase of displacement.

Since the area of the hysteresis loop indicates the energy dissipation in each cycle, these plots give a visual representation of the energy dissipation. Therefore, the energy dissipated by the joints of the CFDST column is comparatively higher than the joint of the square CFST column, because of the improvement of joint total stiffness. In the case of the ribbed joint specimens, ribbed anchorage webs can obviously improve the mechanical performance of the joints, thus can improve the energy consumption ability. When the specimens reached the ultimate state, $h_e = 0.226\text{--}0.321$, while h_e is about 0.1 for the reinforced concrete joints, and h_e is about

Fig. 6.14 Equivalent viscous damping coefficient versus number of half-cycles

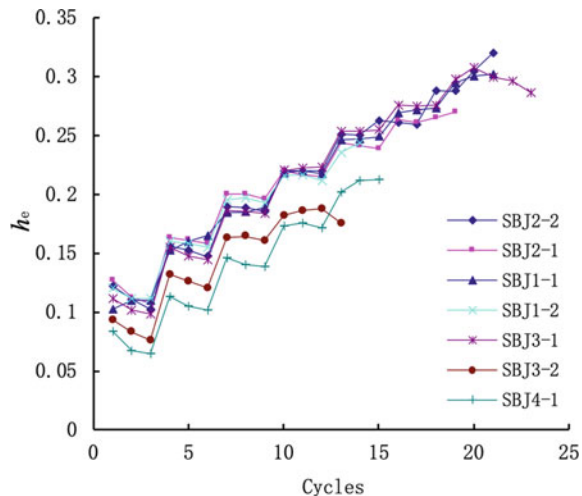
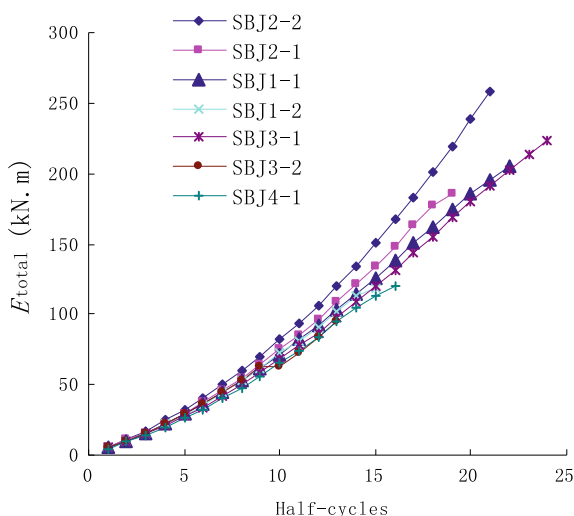


Fig. 6.15 Accumulated energy consumed versus number of half-cycles



0.3 for steel structure joints (Xu and Nie 2011). The equivalent damping viscous coefficients of the joint of the CFDST column increases significantly than reinforced concrete, and is close to the steel structure. Therefore, this new connection has better energy consumption ability, which can completely satisfy the requirements of structure seismic design.

The total dissipated energy E_{total} could also be used to describe the energy dissipation capacity of the joints, which is obtained by accumulating the energy from each half cycle (Nie and Ran 2013). As shown in Fig. 6.15, from the elastic-plastic state, E_{total} continues to significantly increase though the bearing capacity of the specimen grows very slowly. As for Specimens SBJ2-1 and SBJ2-2 with ribbed anchorage web have better energy dissipation capacity than others. This contributes to the fact that ribbed joint specimens have a higher bearing capacity.

6.5 Finite Element Modeling

6.5.1 Establishment of Finite Element Model

(1) Constitutive model of materials

Steel tube

FE simulation was carried out by the ABAQUS program to obtain more practical data to analyze the mechanical behavior of the vertical stiffener joint in the CFDST structure. The stress-strain relationship model proposed by Abdel-Rahman and Sivakumaran (1997) is adopted for the square steel tube column, as shown in Fig. 6.16,

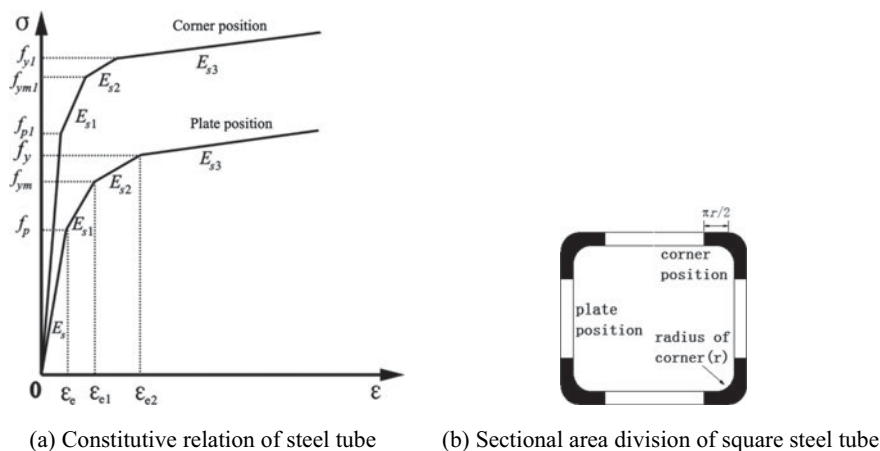


Fig. 6.16 σ - ϵ relation curve of cold-formed steel

the square steel tube plate is divided into two parts: corner position and side position. And the stress-strain relation of circular steel tube is modelled by the equation of plate position of square steel tube. The constitutive relation of steel is shown in Eqs. 6.1 and 6.2.

$$\sigma = \begin{cases} E_s \epsilon & (\epsilon \leq \epsilon_e) \\ f_p + E_{s1}(\epsilon - \epsilon_e) & (\epsilon_e < \epsilon \leq \epsilon_{e1}) \\ f_{ym} + E_{s2}(\epsilon - \epsilon_{e1}) & (\epsilon_{e1} < \epsilon \leq \epsilon_{e2}) \\ f_y + E_{s3}(\epsilon - \epsilon_{e2}) & (\epsilon_{e2} \leq \epsilon) \end{cases} \quad (6.1)$$

$$f_{y1} = f_y \left[0.6 \frac{B_c}{(r/t)^m} + 0.4 \right] \quad (6.2)$$

where: $f_p = 0.75 f_y$, $f_{ym} = 0.875 f_y$, $\epsilon_{e1} = \epsilon_e + 0.125 f_y / E_{s1}$, $\epsilon_{e2} = \epsilon_{e1} + 0.125 f_y / E_{s2}$; B_c and m are the coefficients related to the ultimate tensile strength f_u and the yield strength f_y of steel, $B_c = 3.69(f_u/f_y) - 0.819\left(\frac{f_u}{f_y}\right)^2 - 1.79$, $m = 0.192(f_u/f_y) - 0.068$.

H-shaped steel beam

The simplified trilinear follow-up strengthening model (Zhang et al. 2018) was adopted for H-shaped steel beams and other steel members, as shown in Fig. 6.17. In this figure, $\epsilon_A = f_y / E_s$, $\epsilon_B = (f_u - f_y) / 0.1 E_s$, $\epsilon_C = 0.03$.

Concrete

The CDP (Concrete Damage Plastic) model is needed to input compressive and tensile constitutive of concrete. When the CFDST column is axially compressed, the

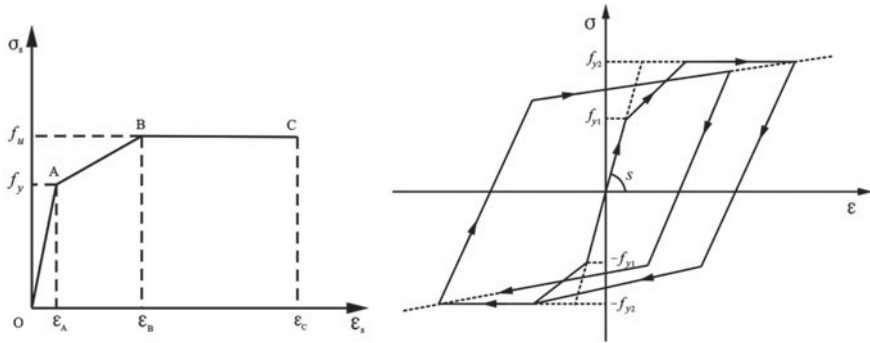


Fig. 6.17 σ - ε relation curve of H-shaped steel beam

core concrete is restrained by steel tubes. There is an interaction between concrete and the steel tube, which makes the mechanical properties of core concrete more complicated. Han et al. (2004, 2016) considered the interaction between the steel tube and the core concrete by introducing the coefficient ξ of constraint effect, and presents the constitutive relation of the core concrete in the steel tube. The stress-strain relation of concrete in the square steel tube and concrete in the circular steel tube is summarized as shown in Eq. 6.3

$$y = \begin{cases} 2 \cdot x - x^2 & (x \leq 1) \\ \frac{x}{\beta(x-1)^\eta + x} & (x \geq 1) \end{cases} \quad (6.3)$$

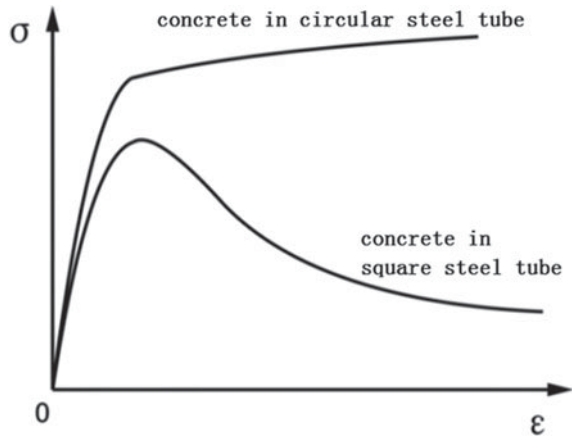
where: $x = \frac{\varepsilon}{\varepsilon_{c0}}$, $y = \frac{\sigma}{\sigma_{c0}}$; $\sigma_{c0} = f'_c$ (N/mm²), σ_{c0} is the peak value of compressive stress for concrete; $\varepsilon_{c0} = \varepsilon_c + 800 \cdot \xi^{0.2} \times 10^{-6}$, ε_{c0} corresponds to the strain at the peak compressive stress; $\varepsilon_c = (1300 + 12.5 \cdot f'_c) \times 10^{-6}$;

$$\eta = \begin{cases} 2 & \text{(Concrete filled circular steel tube)} \\ 1.6 + \frac{1.5}{x} & \text{(Concrete filled square steel tube)} \end{cases}$$

f'_c is the compressive strength of concrete cylinders.

According to the experimental data of concrete properties and the model of confined concrete in the steel tube, the compressive constitutive relation of concrete is shown in Fig. 6.18. The stress-strain model of uniaxial tensile concrete was based on the simplified stress-strain curve recommended by Lu et al. (2009). At the same time, according to the method of measuring the damage coefficient of concrete proposed by Birtel and Mark (2006), the compression damage factor and the tensile damage factor were introduced to simulate concrete. The stress-strain curve equation model of concrete (Lubliner et al. 1989; Lee and Fenves 1998) under uniaxial tension is

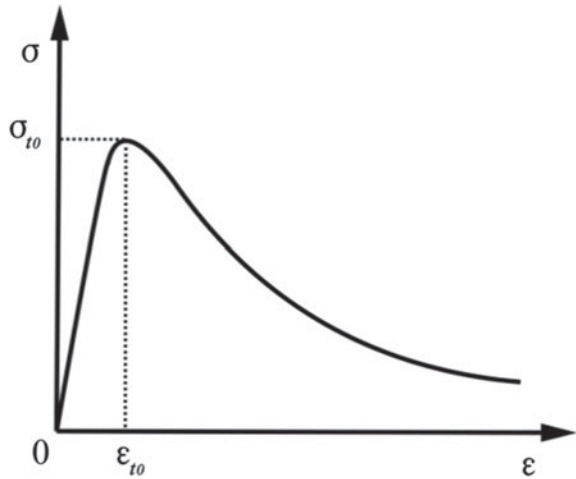
Fig. 6.18 Compressive stress-strain relationship curve of core concrete



$$y = \begin{cases} 1.2 \cdot x - 0.2 \cdot x^6 & (x \leq 1) \\ \frac{x}{0.3 \cdot \sigma_{t0}^2 (x-1)^{1.7} + x} & (x > 1) \end{cases} \quad (6.4)$$

where: $x = \frac{\varepsilon_t}{\varepsilon_{t0}}$, $y = \frac{\sigma_t}{\sigma_{t0}}$; σ_{t0} is the peak tensile stress, ε_{t0} corresponds to the strain at the peak tensile stress, according to Eqs. 6.5a and 6.5b calculations. The tensile stress-strain curve of concrete in this book is shown in Fig. 6.19 according to the *Code for Design of Concrete Structures (GB 50010-2010)*.

Fig. 6.19 Tensile stress-strain relationship curve of core concrete



$$\sigma_{t0} = 0.26 \cdot \left(1.25 \cdot f'_c\right)^{\frac{2}{3}} \quad (6.5a)$$

$$\varepsilon_{t0} = 43.1 \cdot \sigma_{t0}(\mu\varepsilon) \quad (6.5b)$$

Parameter definition of the concrete damage plastic model under cyclic loading

Under cyclic loading, the determination of elastic modulus, Poisson's ratio and material constitutive model in the plastic damage model of concrete is similar to the monotonic load analysis. The main difference is: the model needs to determine the coefficient of tensile and compressive damage (d), coefficient of stiffness restoration (w) under cyclic loading.

- (1) General characteristics of the plastic damage model of concrete under cyclic loading

Because of the action of reciprocating load, the failure surface of concrete is changing constantly, the changing tendency is related to the equivalent plastic tensile strain and the equivalent plastic compressive strain. Therefore, the relationship of σ - ε under tension or pressure can be described as following. Before the tension crack of concrete occurs, the relationship of σ - ε is linear. After the tension crack, the material enters the softening stage, if unload at some point (ε_t, σ_t), the strain of cracking without considering damage is:

$$\tilde{\varepsilon}_t^{ck} = \varepsilon_t - \varepsilon_{0t}^{el} \quad (6.6)$$

The elastic strain is $\varepsilon_{0t}^{el} = \frac{\sigma_t}{E_c}$, where E_c is the initial elastic modulus of concrete. Considering the damage, the tensile plastic strain is

$$\tilde{\varepsilon}_t^{pl} = \tilde{\varepsilon}_t^{ck} - \frac{d_t}{1 - d_t} \frac{\sigma_t}{E_c} \quad (6.7)$$

Tensile stress is

$$\sigma_t = (1 - d_t) E_c \varepsilon_t^{el} = (1 - d_t) E_c (\varepsilon_t - \tilde{\varepsilon}_t^{pl}) \quad (6.8)$$

Effective tensile stress is

$$\bar{\sigma}_t = \frac{\sigma_t}{1 - d_t} = E_c (\varepsilon_t - \tilde{\varepsilon}_t^{pl}) \quad (6.9)$$

where d_t is the coefficient of tensile damage as follows.

$$d_t = \begin{cases} 0, & \text{(no damage)} \\ 1, & \text{(complete damage)} \end{cases} \quad (6.10)$$

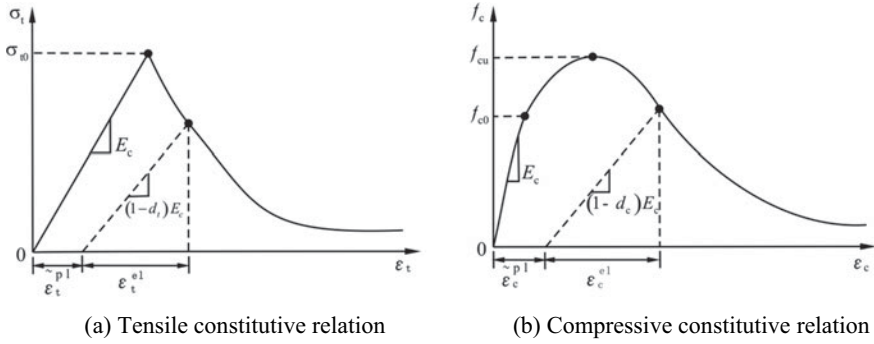


Fig. 6.20 Plastic damage module of concrete

In the ABAQUS process of calculation, the program will be based on the tensile stress at this time and the tensile damage value calculated by Eq. 6.7. The tensile plastic strain considering the damage is calculated by $\tilde{\varepsilon}_t^{pl}$, the value of effective tensile stress is obtained from Fig. 6.20a.

When the compressive stress is no more than a certain value, the curve is linear, then with the increase of the compressive stress of concrete, the material gradually reaches the softening stage. Thus, if it is unloaded at this softening point, the damage value of the cracking strain will not be taken into account as follow.

$$\tilde{\varepsilon}_c^{in} = \varepsilon_c - \varepsilon_{0c}^{el} \quad (6.11)$$

where $\varepsilon_{0c}^{el} = \sigma_c / E_c$. Considering damage, the compressive plastic strain is

$$\tilde{\varepsilon}_c^{pl} = \tilde{\varepsilon}_c^{ck} - \frac{d_c}{1 - d_c} \frac{\sigma_c}{E_c} \quad (6.12)$$

Compressive stress is

$$\sigma_c = (1 - d_c) E_c \varepsilon_c^{el} = (1 - d_c) E_c (\varepsilon_c - \tilde{\varepsilon}_c^{pl}) \quad (6.13)$$

Effective compressive stress is

$$\bar{\sigma}_c = \frac{\sigma_c}{1 - d_c} = E_c (\varepsilon_c - \tilde{\varepsilon}_c^{pl}) \quad (6.14)$$

d_c is the compression damage coefficient.

$$d_c = \begin{cases} 0, & \text{(no damage)} \\ 1, & \text{(completely damage)} \end{cases} \quad (6.15)$$

(2) Definition of damage coefficient (d)

Concrete will produce a series of tension and compression cracks under recycling loads. The development and closure of these cracks and the interaction among them will directly lead to the complexity of the damage mechanism of concrete. The damage index of concrete herein is determined according to the following formulas (Kratzig and Polling 2004; Birtel and Mark 2006).

$$d_c = 1 - \frac{(\sigma_c + n_c \sigma_{c0})}{E_c(n_c \sigma_{c0}/E_c + \varepsilon_c)} \quad (6.16a)$$

$$d_t = 1 - \frac{(\sigma_t + n_t \sigma_{t0})}{E_c(n_t \sigma_{t0}/E_c + \varepsilon_t)} \quad (6.16b)$$

where d_c , d_t are compressive and tensile damage coefficients of concrete, respectively; E_c is the elastic modulus of concrete; σ_c , ε_c are respectively compressive stress and strain of concrete; σ_t , ε_t are respectively tensile stress and strain of concrete; σ_{c0} , ε_{c0} are respectively peak value of compressive stress and corresponding compressive strain of concrete; σ_{t0} , ε_{t0} are respectively peak value of tensile stress and corresponding tensile strain; n_c , n_t are respectively index coefficient of compressive and tensile damage of concrete, according to the results of reference (Li 2011), core concrete in a steel pipe is $n_c = 2$, $n_t = 1$. Then the constitutive relation of concrete subjected to tension and compression is determined. According to (6.16a) and (6.16b), the curve data of damage coefficient for concrete with plastic strain are obtained.

(3) Stiffness recovery characteristics

The experimental results show that the elastic stiffness recovery of concrete under repeated loads is called “the unilateral effect”. Cracks occur when the concrete is under tension, and when the load turns to compression, the previous crack will be closed but new compressive cracks occur. This is an important characteristic of concrete resulting in the restoration of the compressive stiffness.

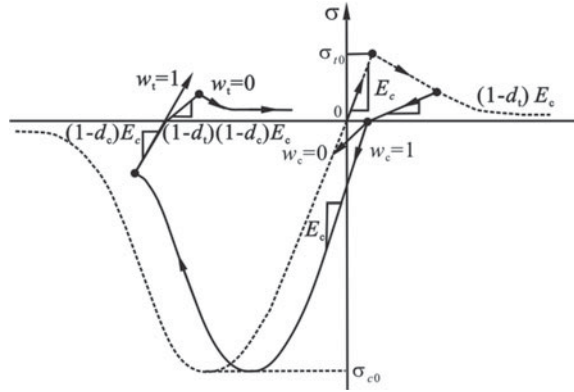
The weighted coefficients of tensile and compressive stiffness restoration are adopted (w_t and w_c) to describe the characteristics of concrete under cyclic loading. ABAQUS assumes

$$(1 - d) = (1 - s_t d_c)(1 - s_c d_t) \quad (6.17)$$

$$s_t = 1 - w_t r^*(\sigma_{11}) \quad (6.18)$$

$$s_c = 1 - w_c (1 - r^*)(\sigma_{11}) \quad (6.19)$$

Fig. 6.21 Restoration of tensile and compressive stiffness for concrete



$$r^*(\sigma_{11}) = H(\sigma_{11}) = \begin{cases} 1 & \sigma_{11} > 0 \\ 0 & \sigma_{11} < 0 \end{cases} \quad (6.20)$$

The weighted coefficients of stiffness restoration w_t and w_c are material properties to influence the tensile and compressive stiffness of concrete under cyclic loading. Figure 6.21 is a schematic diagram of concrete's tensile and compressive stiffness restoration under repeated loads. Refer to the study on concrete-filled steel tubular structures in the monograph (Zhang et al. 2018), that's $w_c = 0.2$, $w_t = 0.0$.

(2) Contact between steel and concrete

Contact between steel and concrete are considered as the bond slip of normal direction and tangential direction. Hard contact was used in the normal direction, while Mohr–Coulomb friction model with the friction coefficient 0.25 in the tangential direction. Shear stress (τ) between the steel tube and core concrete is expressed by

$$\tau = \mu_f \cdot p \geq \tau_{bond} \quad (6.21)$$

where μ_f represents the coefficient of friction, p represents the normal pressure, τ_{bond} represents average bond shear stress. Baltay and Gjelsvik (1990) suggested that μ_f should be between 0.2 and 0.6; Schneider (1998) suggested μ_f can take 0.27; Roeder et al. (1999) and Morishita et al. (1979a, b) conducted the average bond shear stress τ_{bond} for circular and square CFST. It was proved that τ_{bond} decreased with the increase of diameter-thickness ratio of steel tubes. So τ_{bond} (1999) can be calculated by

$$\tau_{bond} = 2.314 - 0.0195(d/t) \quad (6.22)$$

where d is the diameter of the core concrete, t is the wall thickness of the steel tube.

(3) Unit selection and modeling

Both steel and core concrete use the 8-node reduction integral formatted three-dimensional hexahedral element (C3D8R) in finite element modeling. Cell meshing is carried out by structured adaptive meshing. The model used mesh optimization in this chapter. The boundary conditions and loading of the finite element model as shown in Fig. 6.22. The model considers the material and geometric nonlinearities. Two reference points were established on the top and bottom of the column, respectively. An axial load N_0 was applied to the CFDST column top in the first step, and a cyclic displacement load was subsequently applied to the beam ends. The Newton-Raphson method in the ABAQUS standard implicit calculation module was used to solve the model.

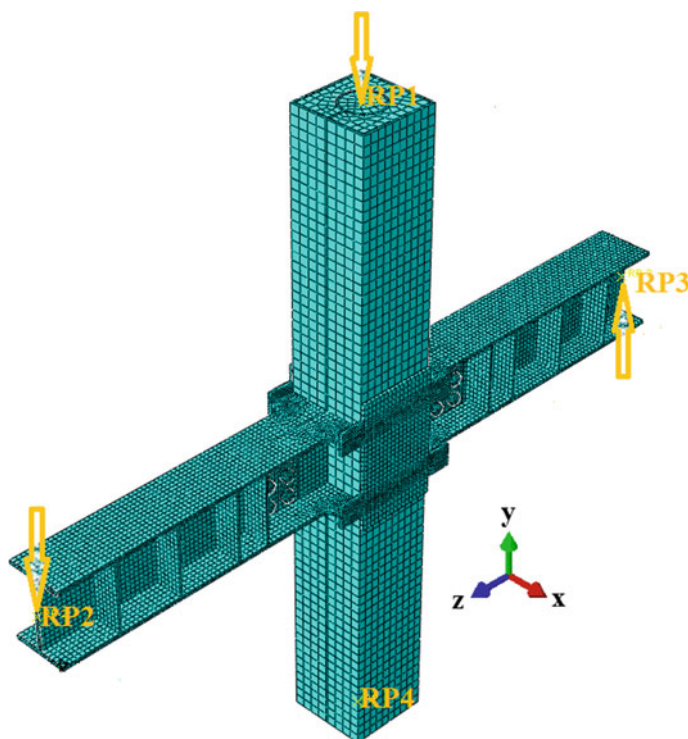


Fig. 6.22 Boundary condition of the FE model

6.5.2 Analysis of Loading Process

Figure 6.23 lists the Mises stress distribution of SBJ1-1 with displacement loading. As can be seen from Fig. 6.23, when the beam end displacement reaches 12 mm, the steel stress at the curvature of the horizontal endplate and the connection between the outer steel tube and the anchorage web is large, with the maximum value of 256.49 MPa. With the increase of loading, the stress of joint core is increasing since the internal force is transferred through the horizontal endplate, anchorage webs and vertical stiffeners. When the beam end displacement is 18 mm, the stress value at the curvature of the horizontal endplate increases obviously to 469.56 MPa. In particular, the steel stress at the edge is the largest, which indicates the ultimate failure mode of plastic hinge points. In the following, the stress value of the horizontal endplate increases continuously, but the increase tends to be slow. When the displacement of the beam end reaches 36 mm, there is a slight buckling deformation at the curvature of the horizontal endplate. As the displacement of beam end increasing, there is a buckling deformation at the anchorage web gradually. As can be seen from the ultimate state diagram of beam end displacement of 60 mm, the horizontal endplates, anchorage webs on both sides are buckled already, but the steel tube in the joint core is still relatively intact under the protection of concrete and vertical stiffeners do not have major deformation.

Figure 6.24 shows the Mises stress distribution of joint SBJ2-2. When the beam end displacement is loaded to 6 mm, the steel stress at the curvature of the horizontal endplate and the connection between the outer steel tube and the anchorage web is larger than unribbed joint SBJ1-1, indicating that the stiffness of ribbed joint is large. As the beam ends displacement is 18 mm and 24 mm, the stress value in the joint core increases continuously, and the stress concentration in the rib is evident. When the beam end displacement is 30 mm, the curvature of the horizontal endplate is slightly buckled and deformed firstly, and the stress value rises gently to 436.45 MPa. As the beam ends continue to be loaded, the stress distribution tends to be stable, and the deformation at the curvature of the horizontal endplate becomes increasingly obvious. The rib of the anchorage web also begins to buckle from the moment when beam end displacement reaches 42 mm. Meanwhile, the buckling deformation of the anchorage web gradually increases until the beam end displacement reaches 66 mm to the final failure state.

As can be seen in Figs. 6.23 and 6.24, the vertical stiffeners are in the state of pure shear stress during the whole loading process. Apart from some stress concentration phenomena in the ribbed anchorage web, vertical stiffeners and steel tubes in panel zone are basically in the elastic state. The vertical stiffeners are directly welded to the steel tube webs, which can effectively share the shear force at the basis of protecting the steel tube and the horizontal endplates (Zhang et al. 2018). By comparing the stress distribution in Figs. 6.23 and 6.24, the vertical stiffener together with the anchorage web played a dominated role in the internal force-transferring mechanism, but the stress in vertical stiffeners of the ribbed joint is larger than the unribbed joint at the same beam end displacement because the ribbed anchored web improves the transmission of stress.

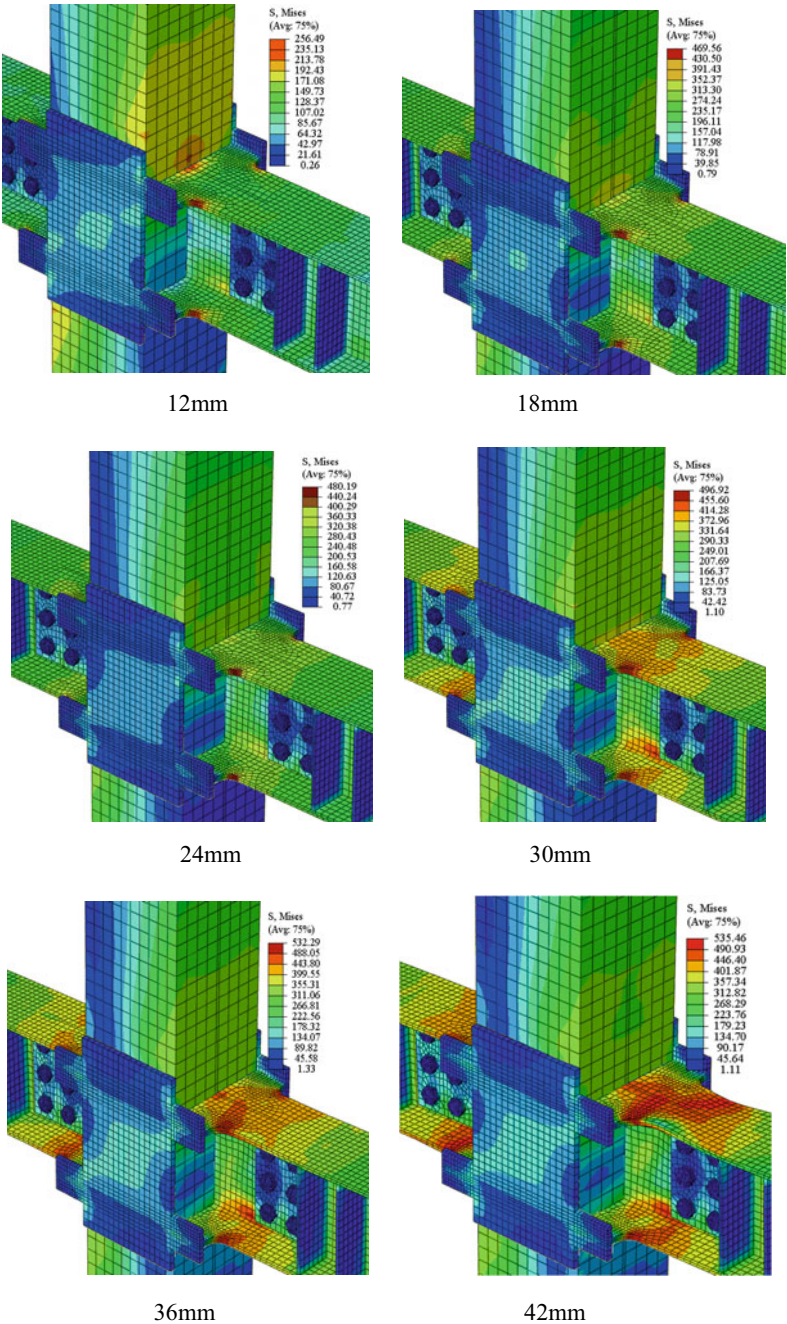


Fig. 6.23 Stress distribution during the loading process (SBJ1-1)

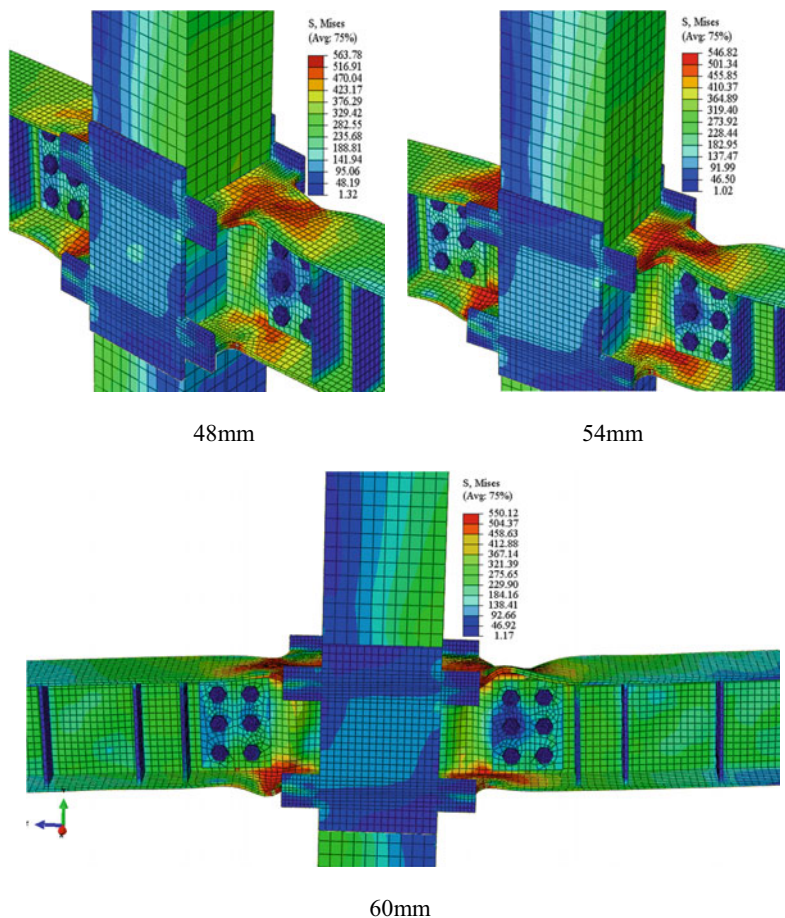


Fig. 6.23 (continued)

6.5.3 Verification of Finite Element Modeling

(1) Comparison of hysteresis loops

Figure 6.25 shows finite element simulated hysteresis loops and experimental ones of west beam end of the joint specimen. As can be seen from Fig. 6.25, the hysteresis loop of the finite element simulation is in good agreement with the experimental hysteresis loop, which embodies the complete elastic, elastic-plastic and plastic failure phase. The model in the finite element simulation are ideal, it ignores the influence of the welding at the horizontal endplate and the H-shaped beam. So, the load value of the simulated hysteresis loop is slightly higher than the test ones, but the error is less than 10%. The stiffness of the finite element model is larger than that of the experimental

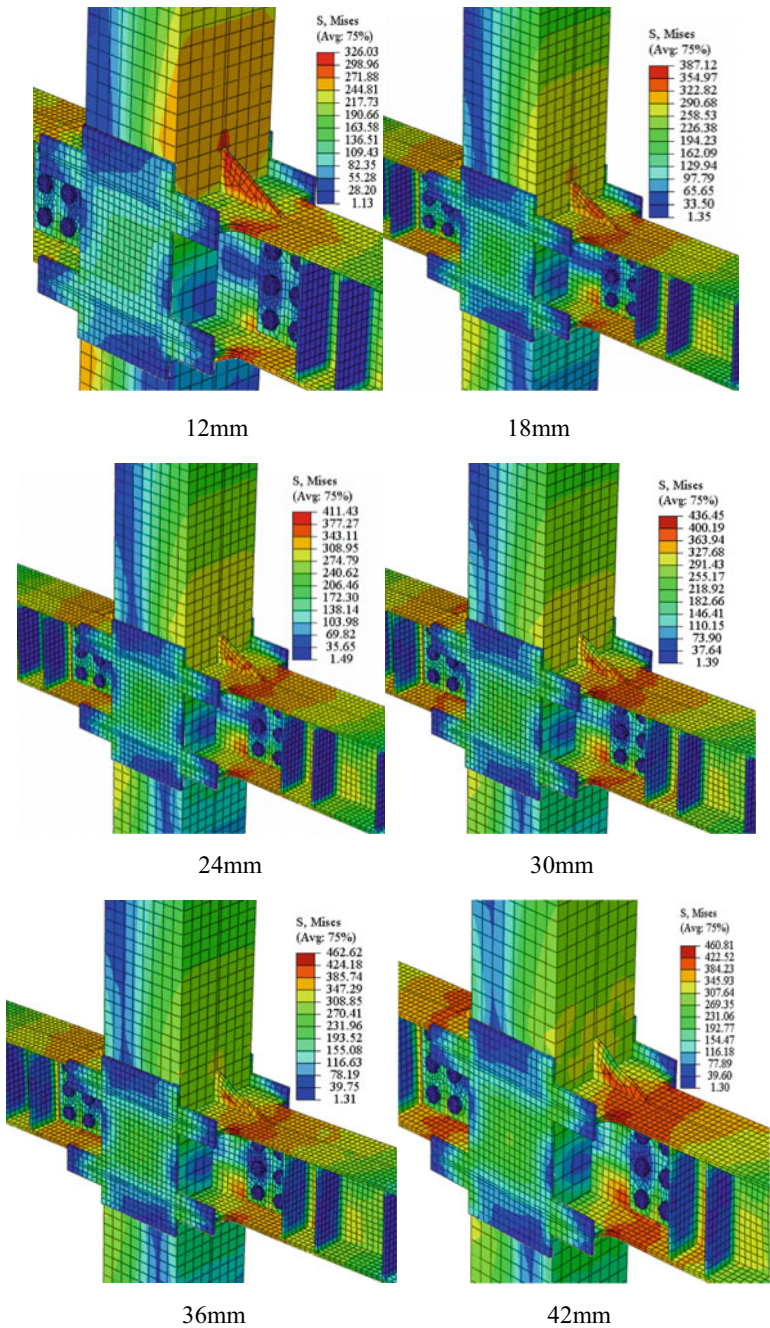


Fig. 6.24 Stress distribution during the loading process (SBJ2-2)

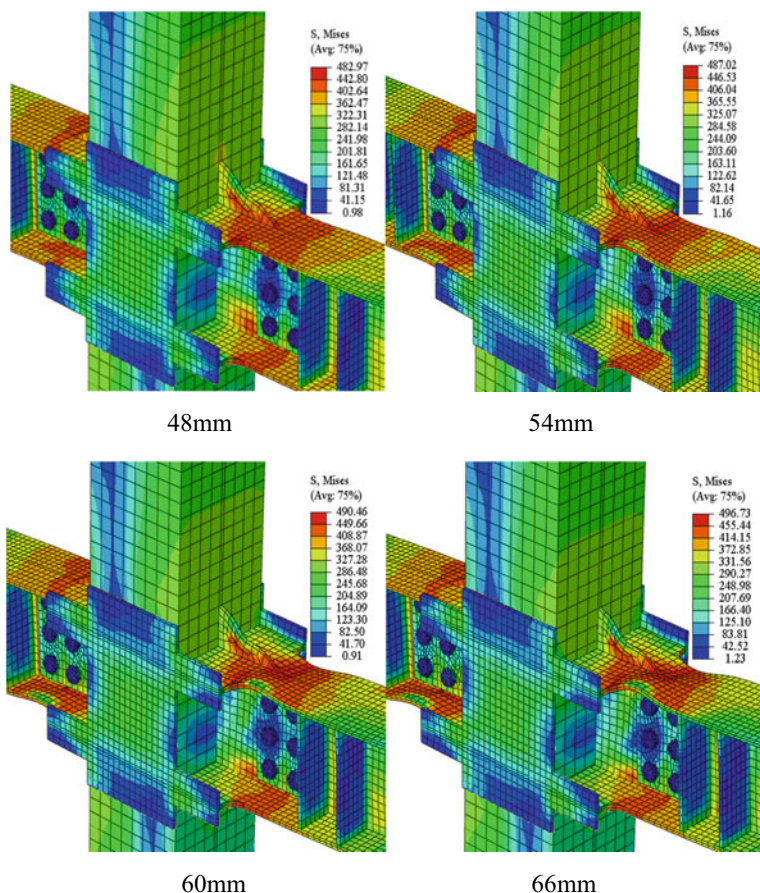


Fig. 6.24 (continued)

model, this is due to the fact that weld of the specimen is basically on the steel beam, but the influence of welds is not considered in the finite element model. The hysteresis loops of these joints are full, showing obviously spindle-shaped, and having good seismic performance.

(2) Comparison of destruction of typical specimens

Figures 6.26 and 6.27 show the failure modes of test and simulation. It can be concluded that the failure modes of the joints are generally consistent. The plastic hinges were observed on the beam flange close to the horizontal endplate. As a result of numerical simulation failed to simulate the tearing phenomenon of steel, it can be considered a large deformation of steel where the buckling occurred. In conclusion,

the accumulated plastic deformation, local buckling, and joint core stress distribution are well simulated. As predicted, as a relatively strong panel zone, limited inelastic deformation could develop before significant flexural yielding developed in the beam section, and the joint specimens failed at the plastic hinges in the vicinity of the arc section of the horizontal endplates. Therefore, the design of the vertical stiffener joint is in line with the seismic principle of “strong shear and weak bending”.

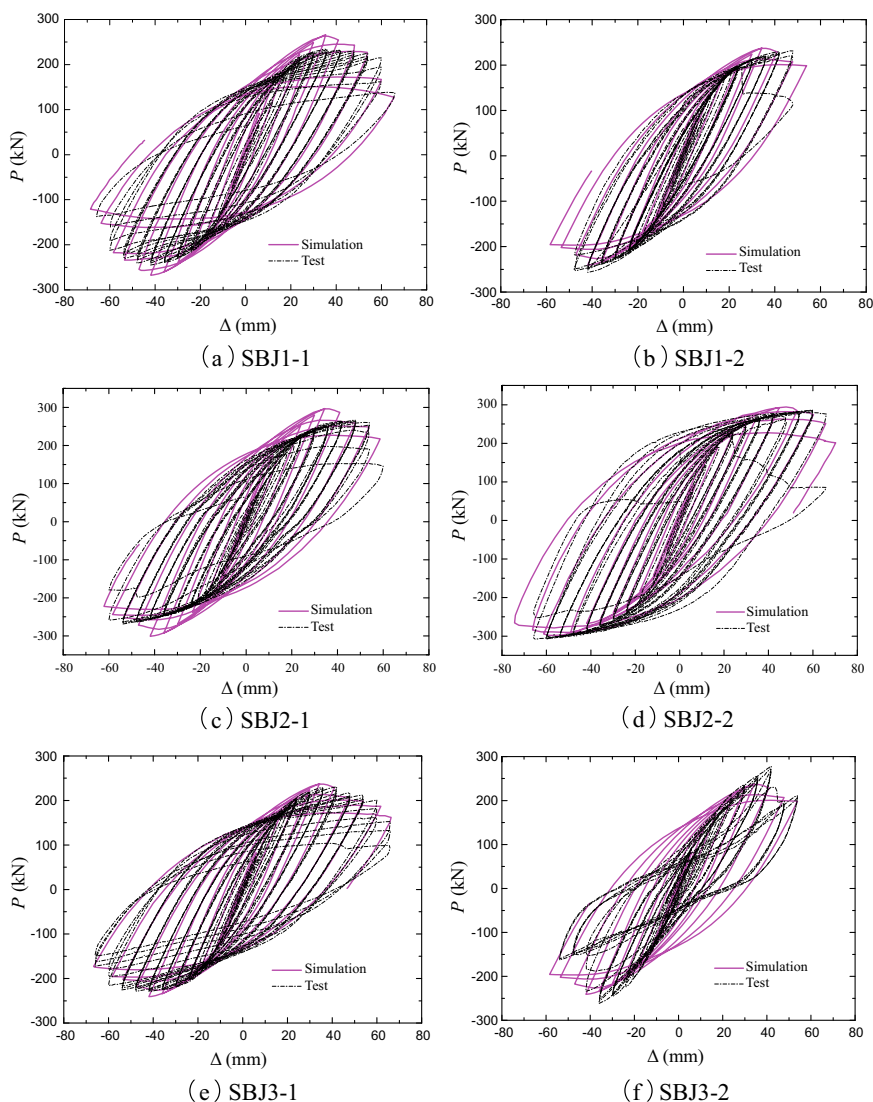
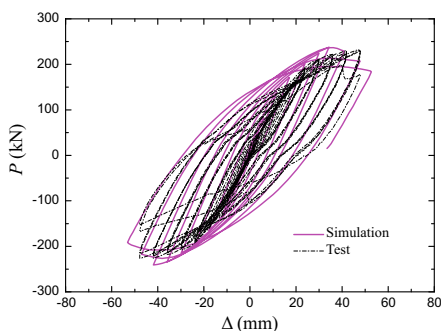


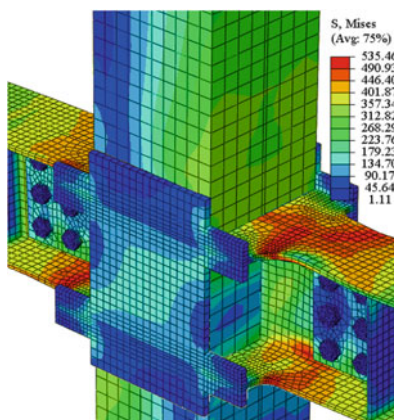
Fig. 6.25 Comparison of hysteresis loops



(g) SBJ4-1

Fig. 6.25 (continued)

(a) Test



(b) Simulation

Fig. 6.26 Failure mode of the joint without ribs

6.6 Calculation of Bearing Capacity

6.6.1 *Shear Force Transfer Model and Shear Resistance in Panel Zone*

In FE modeling, the capacity of the various elements of the connection detail can be predicted. Shear force in panel zone is transmitted by the interaction of the outer steel tube wall, vertical stiffeners, anchorage webs, and the core concrete as shown in the shear stress distribution in Fig. 6.28. As can be seen, vertical stiffeners have great contribution in shear resistance at the maximum load. In contrast with the unribbed joint, the rib of the anchorage web has the stress concentration to reduce the

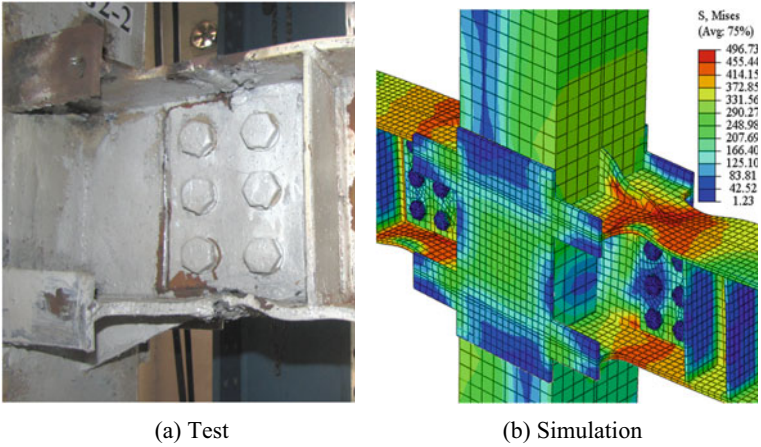


Fig. 6.27 Failure mode of the joint with ribs

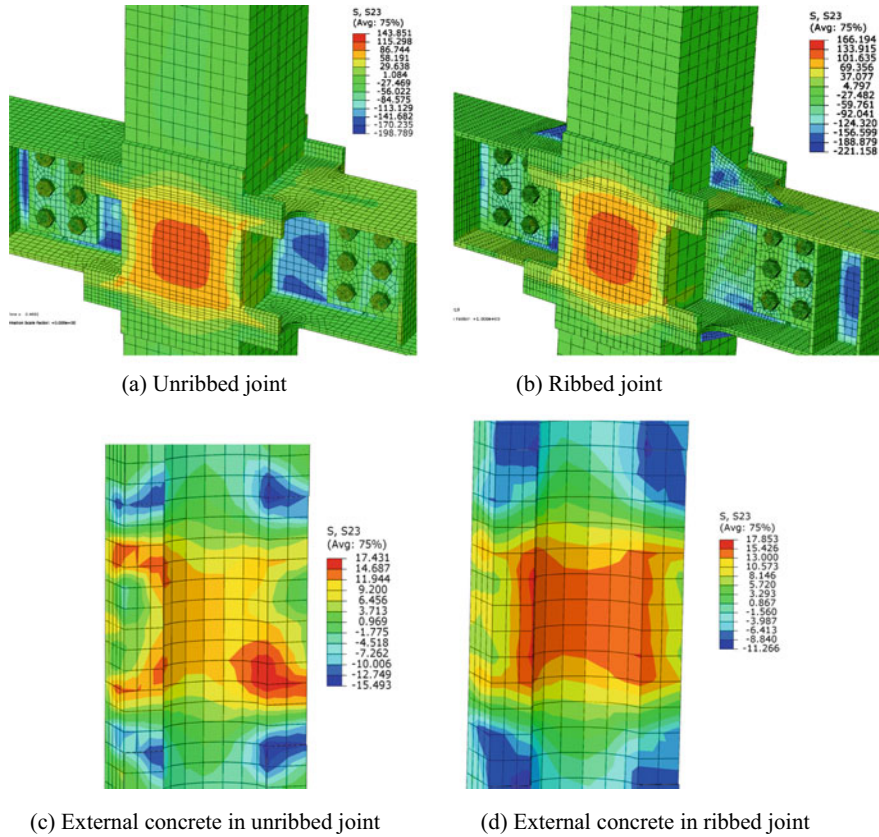


Fig. 6.28 Shear stress distribution at the maximum load

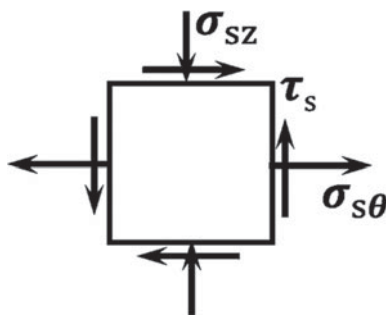
magnitude of concrete's shear stress as shown in Fig. 6.28d. Beyond that, the shear stress distribution in external concrete between the double steel tubes is basically similar for both unribbed joint and ribbed joint. The steel tubes also have the same principle of the shear stress distribution as concrete. Besides, the stress in materials at the peak load has not reached the yielding shear stress yet, so the failure mechanism of the vertical stiffener joint should not happen in the panel zone. After this investigation, we can get the possible exact horizontal shear force in panel zone supported by the main components of the vertical stiffener joint (Zhang et al. 2019; Zhou et al. 2019).

The calculation formula for the CFST structure proposed by the Architectural Institute of Japan (1987) considers the shear strength of both steel tube and infilled concrete. Thus, the shear resistance of the vertical stiffener joint in the CFDST structure contains the contribution from double steel tubes, force-transferring connectors, and core concrete. Among the contributions, concrete provides the shear bearing capacity in the diagonal strut model under the theory of plane shear state. Shear resistance of each component takes into account every stress state, and it can be computed by analyzing different deformation mechanism, then the ultimate shear capacity of the joint will be obtained on the principle of limit equilibrium superposition. The validity and practicability of the strength superposition method for estimating shear strength were verified explicitly by many researchers (Chen et al. 2009; Oinonen and Marquis 2013; Pinho Ramos et al. 2014; Ma et al. 2018). Therefore, the shear resistance of each component is analyzed separately as following.

Steel tube

Since the confining effect of the concrete-filled square steel tube is complex, the square tube is firstly equivalent to a circular steel tube using the method of the equal area (Zhang et al. 2018). Under the axial compressive force on the column, the circumferential tensile stress will decrease the shear capacity of the steel tube, so the lateral tensile stress $\sigma_{s\theta}$ of the steel tube is considered in the stress state as shown in Fig. 6.29, where σ_{sz} stands for the compressive stress under axial compression, τ_s is the shear stress of the steel tube. They can be expressed as: $\sigma_x = \sigma_{s\theta}$, $\sigma_y = \sigma_{sz}$, $\tau_{xy} = \tau_s$, then the principal stresses are described as

Fig. 6.29 Stress state of the steel tube



$$\begin{aligned}
\sigma_1 &= \frac{\sigma_{oz} + \sigma_{o\theta}}{2} + \sqrt{\left(\frac{\sigma_{oz} - \sigma_{o\theta}}{2}\right)^2 + \tau_o^2} \\
\sigma_2 &= 0 \\
\sigma_3 &= \frac{\sigma_{oz} + \sigma_{o\theta}}{2} - \sqrt{\left(\frac{\sigma_{oz} - \sigma_{o\theta}}{2}\right)^2 + \tau_o^2}
\end{aligned} \tag{6.23}$$

Yield stress f_{sy} of the steel tube under Von Misses yield criterion is

$$f_{sy} = \sqrt{\sigma_{sz}^2 + \sigma_{s\theta}^2 - \sigma_{sz}\sigma_{s\theta} + 3\tau_s^2} \tag{6.24}$$

So, the shear stress τ_s of steel tube can be deduced as

$$\tau_s = \frac{1}{\sqrt{3}} \sqrt{f_{sy}^2 - \sigma_{sz}^2 - \sigma_{s\theta}^2 + \sigma_{sz}\sigma_{s\theta}} \tag{6.25}$$

Then at the ultimate state of the joint, the ultimate shear resistance V_s of steel tube webs can be obtained as follows based on the tri-linear shear model (Fukumoto and Morita 2000; Nishiyama et al. 2004) as shown in Fig. 6.30.

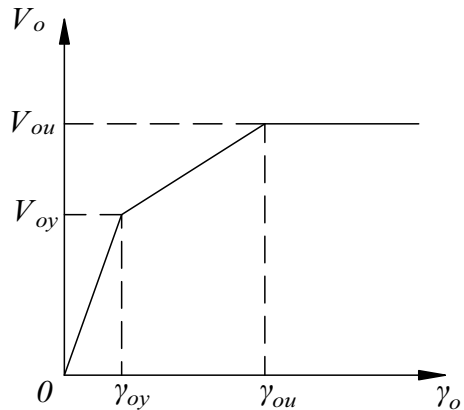
$$V_s = \frac{A_s}{\sqrt{3}} \sqrt{f_{su}^2 - \sigma_{sz}^2 - \sigma_{s\theta}^2 + \sigma_{sz}\sigma_{s\theta}} \tag{6.26}$$

where A_s represents the cross-sectional area of the steel tube web, and f_{su} represents the ultimate shear strength of the steel tube.

Vertical stiffener

The pure shear stress state of the vertical stiffener is $\sigma_x = 0$, $\sigma_y = 0$, $\tau_{xy} = -\tau_v$, then the principal stresses are: $\sigma_1 = \tau_v$; $\sigma_2 = 0$; $\sigma_3 = -\tau_v$. Yield stress f_{vy} under Von

Fig. 6.30 Trilinear shear force-deformation model



Misses yield criterion of the vertical stiffener is $f_{vy} = \sqrt{3}\tau_{vy}$. Similarly, ultimate shear resistance V_v of the vertical stiffener is

$$V_v = \frac{f_{vu}A_v}{\sqrt{3}} \quad (6.27)$$

where A_v denotes the horizontal cross-sectional area of the continuous vertical stiffener; f_{vu} is the ultimate strength of the vertical stiffener.

Anchorage web

Similar to the vertical stiffener, the anchored web is also in a pure shear state, and the ultimate shear resistance V_a is calculated by

$$V_a = \frac{f_{au}A_a}{\sqrt{3}} \quad (6.28)$$

where A_a is the shear area of the anchorage web. For unribbed anchorage webs, it only takes the horizontal cross-sectional area between the double tubes, while the whole horizontal cross-section for the ribbed one.

Joint core concrete

The external and internal core concrete are confined by steel tubes, so the shear resistance of concrete can be taken as a constant value after reaching the maximum shear strength as shown in Fig. 6.31, where γ_p and τ_p represent the peak shear strain and stress respectively, $\tau_p = 0.42f_{cu}^{0.55}$ (Zhang and Guo 1992), f_{cu} is concrete's ultimate compressive strength. The shear resistance V_c of core concrete is calculated by

$$V_c = \tau_p(A_{co} + A_{ci}) \quad (6.29)$$

where A_{co} , A_{ci} respectively, represent the external and internal concrete cross-sectional area.

6.6.2 Moment Transfer Model and Bending Resistance

To get a better view, corresponding nephograms of the vertical stiffener and the anchorage web are used to analyze the moment transfer mechanism. Maximum principal stress distribution of the vertical stiffener in the yielding stage is shown in Fig. 6.32. It indicates that the upper overhang on the left is under tension basically, while the lower one is under compression. In terms of force transfer path, the tensile force exerted on the steel beam flanges is firstly transmitted from the endplate to the overhang of the vertical stiffener, then to the web of the outer steel tube, eventually to

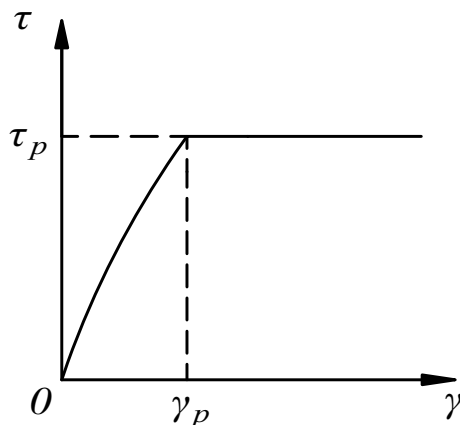


Fig. 6.31 Shear stress-strain curve of the core concrete

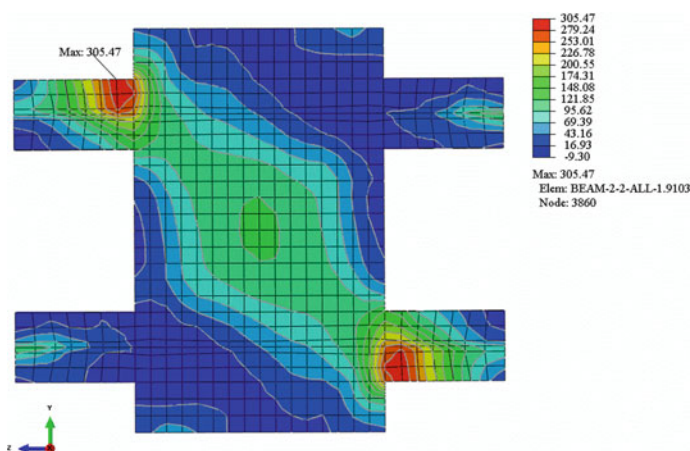


Fig. 6.32 Maximum principal stress distribution of vertical stiffener

the core region of the joint via the anchorage web. Accordingly, because the bending moment can be transformed into a couple at the end of steel beam, the upper and lower beam flanges are acted upon by tensile force and compressive force respectively (Zhang et al. 2015, 2019). The Mises stress distribution of the anchorage web at the maximum load is specified as shown in Fig. 6.33. The anchorage web like a small cantilever slab transmits the shear force and bears the tension passing from the beam flange. Then the anchorage web is capable of passing the tension exerted on itself to the inner tube, making the joint core zone an integrated part. As can be seen in Fig. 6.33, the stress in the anchorage web embedded between double steel tubes is small, and the stress value at the connection with the upper and lower flange of the

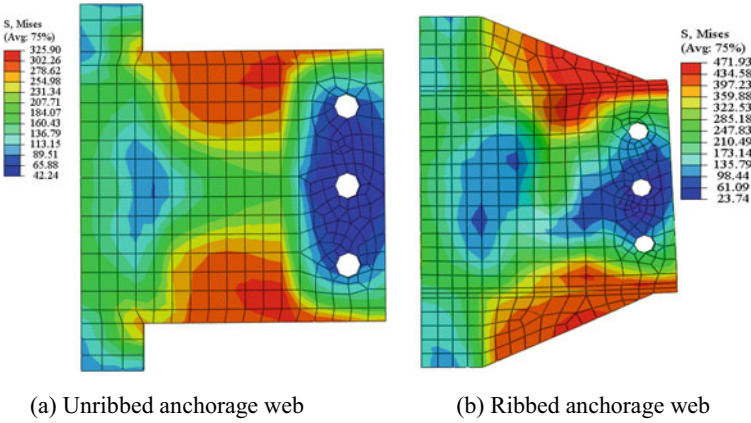
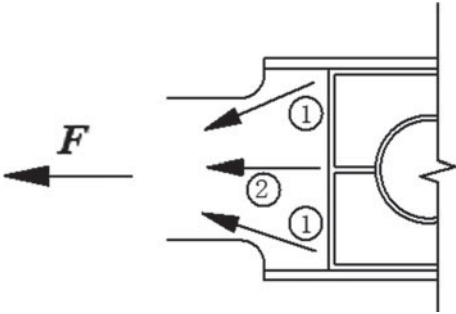


Fig. 6.33 Mises stress nephograms at maximum load

steel beam is large. The advantage of ribbed anchorage web is that the stiffening rib can firstly concentrate the internal force on itself, then transfer force to the double steel tubes. As the overall bearing capacity of the ribbed joint is higher than that of unribbed one, the stress value of the ribbed anchorage web is also larger.

FEM analytical models demonstrate that the vertical stiffener together with the anchorage web can transmit axial internal force of the beam flange through the horizontal endplate. Since bending moment is transmitted from beams to columns in the form of tensile and compressive beam flange, the connection performance and working capability of the beam flange, especially the one in tension, which will directly determine the entire bearing capacity of the joint, is of vital importance to the mechanical properties in the joint zone (Yuen and Kuang 2015). If the tensile flange can transmit the axial force successfully, the transmission of axial force in the compressive flange is also satisfied, so the tension model as shown in Fig. 6.34 is adopted to discuss the problem of transmission mechanism and bearing capacity of the joint. Direction 1 in Fig. 6.34 stands for the tension F_1 of flange transmitted by vertical stiffener, and direction 2 stands for the horizontal resultant force F_2 assumed

Fig. 6.34 Tension model



by the column flange and anchorage web. In this new type of joint, the force in the flanges is directly passed on from the horizontal endplate to the vertical stiffener. On the premise of a guaranteed weld and according to the static equilibrium as well as the ultimate strength of the steel, the calculation of the ultimate tension assumed by the vertical stiffener F_1 can be expressed as follows.

$$F_1 = f_{vu} t_s h_s \quad (6.30)$$

where t_s is the thickness of the vertical stiffener; h_s is the vertical height of the overhang of the vertical stiffener; We ignore the contribution of the concrete for its relatively poor tensile strength and the contribution of the steel tube web for its out-of-plane stress, then the ultimate tension F_2 from the ribbed anchorage web can be calculated by

$$F_2 = f_{au} t_a h_a \quad (6.31)$$

where t_a is the rib thickness; h_a is the rib height of the anchorage web. The transmission of the unribbed anchorage web is supposed to be halved. Finally, on the premise of the equivalent tension and compressive force, the ultimate bending moment can be calculated by

$$M_{uC} = F H_b = (2F_1 + F_2) H_b \quad (6.32)$$

where H_b denotes the beam height.

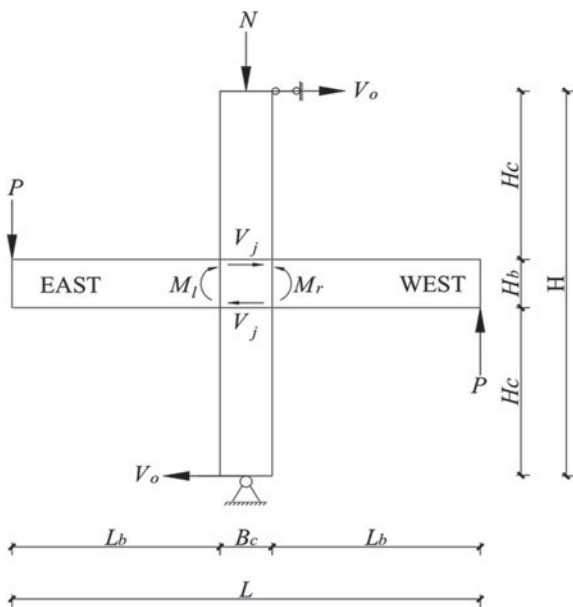
6.6.3 Validation and Analysis

According to the theoretical and experimental data of six joint specimens, the ultimate shear capacity in panel zone and ultimate bending moment of the vertical stiffener joints are listed in Table 6.5. The ultimate shear capacity of the joint was computed by the principle of limit equilibrium superposition. Based on the aforementioned shear capacity equations of the joint components, the shear load bearing capacity V_{uC} is obtained by the Eqs. 6.26–6.29 to assess the cracked vertical stiffener joint with shear failure in the panel zone. M_{uC} is the computed bending resistance obtained from Eq. 6.32. V_{uE} represents the ultimate shear force in panel zone at the ultimate load in the test. M_{uE} represents the ultimate bending moment at the beam plastic hinge corresponding to the average peak value of east and west load in the test.

When the beam-to-column assemblies are subjected to an axial force N on the column and a quasi-static cyclic loading P at beam ends, the internal force diagram in panel zone is shown in Fig. 6.35, where V_j represents the horizontal shear force in panel zone; L_b is the net beam length on one side; H is the calculation height of the

Table 6.5 Calculation results

Specimens	l_O (mm)	l_p (mm)	M_{uE} (kN m)	M_{uC} (kN m)	V_{uE} (kN)	V_{uC} (kN)	$\frac{M_{uC}}{M_{uE}}$	$\frac{V_{uC}}{V_{uE}}$
SBJ1-1	80	900	214.5	208.0	1703.9	4070.9	0.97	2.28
SBJ1-2	120	860	226.0	208.0	1774.2	4544.6	0.92	2.45
SBJ2-1	80	900	246.3	240.9	1959.5	4841.8	0.98	2.37
SBJ2-2	120	860	263.0	240.9	2126.1	5315.5	0.92	2.41
SBJ3-1	80	900	210.1	208.0	1669.5	4070.9	0.99	2.32
SBJ3-2	80	900	250.9	240.9	1856.6	4544.2	0.96	2.60

Fig. 6.35 Force diagram of the testing joint

whole column. According to the equilibrium condition, the horizontal shear force in the panel zone can be calculated by the method (Tang 1989) as follows.

$$V_j = \left(\frac{M_l}{H_b - t_b} + \frac{M_r}{H_b - t_b} \right) - V_o = \frac{2PL_b}{H_b - t_b} - \frac{PL}{H} \quad (6.33)$$

where V_o represents the reactive force at the horizontal roller; t_b represents the thickness of the beam flange; M_l , M_r are respectively the bending moment at left and right beam ends. Then V_{uE} is the shear force corresponding to the average maximum quasi-static cyclic load P_{max} at both east and west beam ends. According to Figs. 6.26 and 6.27, the bending moment at the plastic hinge, which can be calculated by

$$M_{uE} = P_{max}l_p \quad (6.34)$$

where l_p stands for the distance from beam end to overhang of the vertical stiffener.

As can be seen in Table 6.5, the calculation results of ultimate shear resistance V_{uC} are significantly larger than those obtained from the test data V_{uE} . The reason is that V_{uC} is the computed shear resistance corresponding to the assumed shear failure, which was computed by the conceptual model for assessing the cracked panel zone under shear failure. But both test and FEM verified that the failure mode of the joint is bending failure, so V_{uE} is the shear force corresponding to experimental bending failure. Though the theoretical shear resistance was not checked via joints of full shear strength because the bending resistance limited the maximum internal shear force in panel zone, it indicates that the joint has sufficient shear storage capacity to avoid the brittle failure, and the mean value of V_{uC}/V_{uE} is 2.41. Therefore, shear resistance in the panel zone provides a large amount of ductility and extra storage capacity so as to improve the seismic performance. On the other hand, the ultimate bending resistance between calculations and experimental results are very close, and the mean value of M_{uC}/M_{uE} is 0.96. Therefore, the tension model for moment transfer is featured with a favorable accuracy, and it can be applied in predicting and evaluating the bending resistance of vertical stiffener joints.

6.6.4 Discussions

Based on the numerical results and general failure modes, the transfer mechanisms of internal forces of the vertical stiffener joint were closely investigated herein, and the bearing capacity computed by the conceptual model was compared with the test results. Next, in order to guarantee the ductile failure, the structural elements should be designed in the future engineering application according to the different contribution to the shear capacity and bending resistance.

In Table 6.5, by comparison of the specimens with overhang length 120 and 80 mm, increasing the overhang length of the vertical stiffener hasn't improved the bending resistance effectively, but the plastic hinge mostly depends on the position of the vertical stiffener, so it can protect the joint core to ensure the strong beam-to-column connection. Meanwhile, lengthening the vertical stiffener's overhang increases the joint's shear resistance to have more storage capacity to avoid the brittle shear failure in panel zone. Hence, the vertical stiffener determines the plastic hinge so as to ensure a higher bearing capacity and stiffness because the joint core zone is further effectively protected.

In the calculation of shear resistance of the vertical stiffener joint, axial compression ratio was taken into consideration for only the steel tubes, while not considered in the calculation of bending resistance. For Specimen SBJ3-1, axial compression

ratio is 0.4. The shear force and bending resistance obtained in the test have no evident changes from Specimen SBJ1-1. Therefore, the calculation method of vertical stiffener joints under the different axial load level in this study sounds reasonable, and axial compression ratio has a little influence on the joint's shear and bending resistance.

Concerning the behavior of the ribbed joints SBJ2-1 and SBJ2-2, the presence of ribbed anchorage web enlarges the internal lever arm, thus increasing the depth of the panel and reducing the actual shear stress. Ribbed anchorage web is an effective way of increasing the bending resistance, and it also improves the computed shear resistance corresponding to the assumed shear failure. But when the overhang is short as 80 mm for SBJ2-1, tearing developed on the connected steel tube under tension and local buckling developed on the relatively thin rib under compression as shown in Fig. 6.9h.

In comparison with the joint specimens SBJ2-1 and SBJ3-2, different thickness of outer steel tube influenced the shear resistance of vertical stiffener joints for the tube web was the main shear component in the joint core. And at the condition of strong panel zone, the tube's thickness has no effect on the bending resistance. While it deteriorated the foregoing tearing of steel tube, which connected with the rib, so improving the local thickness of the steel tube in joint core is appropriate to improve the joint performance.

6.7 Summary

This chapter presented the vertical stiffener joints between the CFDST columns and H-shaped steel beams, which can be considered as a typical rigid connection. The CFDST column and H-shaped steel beams were connected by the vertical stiffeners and embedded anchorage webs. Six CFDST column-to-beam joints and a square CFST column-to-beam joint were tested. The desired failure mode was a plastic hinge in the steel beam. High quality welding procedures allow a frame specimen to be exercised well beyond elastic limits in order to obtain sufficient ductility. In terms of maximum bearing capacity, stiffness, ductility, energy dissipation capacity and stiffness degradation, the connection assemblages of CFDST columns, as expected, perform better than the square CFST column. Test results demonstrate that the new type joint has the advantage of transferring force reliably, simple construction details, thus it has a wider foreground in CFDST structures.

A finite element analysis (FEA) model of the joint was also established and validated by comparing its predictions with experimental results. Based on finite element modeling (FEM), the internal force transfer mechanism of vertical stiffener joints was analyzed. Analytical models of shear force and bending moment were established through the appropriate material constitutive equations and equilibrium theory.

Then the proposed models were used to predict and evaluate the shear and bending resistance of the vertical stiffener joint. The computed bending resistance obtained by the tension model agreed well with the measured experimental data, and the shear resistance in the panel zone was sufficient to guarantee the ductile failure in the test.

Lengthening the overhang of the vertical stiffener will increase shear resistance of the connection to protect the panel zone. The vertical stiffener determined the place of plastic hinge so as to ensure the strong connection between the CFST column and the steel beam. Axial compression ratio has a little influence on the joint's shear and bending resistance, while installing ribbed anchorage webs is an effective way of increasing the bearing capacity. Improving the local thickness of the steel tube in the joint core is appropriate.

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Chapter 7

External Diaphragm Joints to Steel Beams



Abstract A large number of external diaphragm joints in CFST structures have been tested under a variety of conditions. For example, Wang et al. (2008) and Zhang et al. (2012) conducted a pilot study of eight external diaphragm joints under reciprocating loads; the other experiments conducted by Li et al. (2009), Quan et al. (2017), Shin et al. (2008), Wu et al. (2016), etc., showed that the external diaphragm joint was more feasible than other types of joints in CFST structures, and it could easily meet the design requirements of rigid joints. The current development of the external diaphragm joint in steel composite structure was also introduced by some researchers (Fadden et al. 2015; Vulcu et al. 2017; Sabbagh et al. 2013). The main contents were to (1) prove the feasibility of using an external diaphragm between a steel beam and a rectangular hollow section steel tube, (2) evaluate connection detailing requirements in hollow cross-section structures, (3) conduct parametric studies of the moment-resisting joints with external diaphragm collar plates using ABAQUS. To conclude, external diaphragm joints have relatively large stiffness, good plasticity, and high bearing capacity, so it could be applied in the CFDST structure. The external diaphragm joint is improved in terms of the cross-sectional feature of CFDST column in this book, and new test data pertaining to the mechanical behavior of the external diaphragm joint subjected to a cyclically increasing lateral load and a constant axial load was presented. Axial compression ratio of the CFDST column, width of the external diaphragm, configuration of the anchorage web and beam-to-column bending stiffness ratio are considered as the experimental parameters. Moreover, a new optical non-contact technique, digital speckle correlation method (DSCM), was used to measure and observe the joint in the low cycle reciprocating loading test. Further considerations were focused on the strain field in the panel zone and relative beam-to-column rotation, including the classification of joint stiffness.

7.1 Description of the Connection System

7.1.1 Design of New External Diaphragm Joints

The external diaphragm joint has been improved in consideration of the characteristics of the inner and outer steel tubes in a CFDST column (Zhang et al. 2019, 2020). The external diaphragm is still served as the primary force transfer member. Besides, there are anchored webs embedded between the double tubes to withstand shearing force and bending moment from the steel beam. Both the AIJ standard for structural calculation of reinforced concrete structures (2001) and technical code for concrete-filled steel tubular structures in China (GB 50017-2003 2003; CECS 28: 2012 2012; DL/T 5085-1999 1999) contain specified design formulas and structural requirements for external diaphragm joints to tubular columns. The details of the joint configuration are shown in Fig. 7.1. As shown in Fig. 7.1a, the upper and lower octagonal external diaphragms are located outside the column and welded to the outer steel tube. Two ends of the external diaphragm connect with the steel beam flanges through butt welds. The anchored web penetrates the outer tube and welds on the inner tube. The other side is bolted to the steel beam web using connecting plates, as shown in Fig. 7.1b. A ribbed joint is designed to strengthen the connection, as shown in Fig. 7.1c. Figure 7.1d, e show the structural detail of Specimen SRJ6-1 with a through-web, where a single square CFST column is chosen.

In a real engineering structure, joints play a key role in determining whether a structure will reach its theoretical ultimate load, because plastic hinges usually occur at several members, thus there are two types of joints to be tested in this study. Five joint specimens named SRJ1-1, SRJ1-2, SRJ1-3, SRJ2-1, and SRJ3-1, were designed to cater for the seismic requirements of “strong column & weak beam”. This design was chosen to provide a relatively strong panel zone so that significant flexural yielding can develop in the beam section. Four joint specimens named SRJ4-1, SRJ4-2, SRJ5-1, and SRJ5-2, were designed to exhibit “strong beam & weak column” behavior and the dominant column failure mode can be expected. The total height of the column is 2000 mm and the total beam length is 3700 mm. Geometric properties and characteristics of specimens are shown in Table 7.1, where h , b_f , t_w , and t_f are, respectively, the overall height, overall width, web thickness and flange thickness of the I-shaped beam. The joint specimens were designed to investigate the effects on changing axial load of the CFDST column, width of the external diaphragm, configuration of the anchored web and beam-to-column bending stiffness ratio.

- *Axial load level of the CFDST column:* axial compression ratio $n = 0.248, 0.361$ and 0.567 . $n = N/N_u$, the calculation of N_u refers to Sect. 6.3.
- *Width of the external diaphragm:* there are three types of external diaphragms according to their width. The width of the external diaphragm is determined by the Japanese code AIJ (2001) and the technical specification for CFST structures

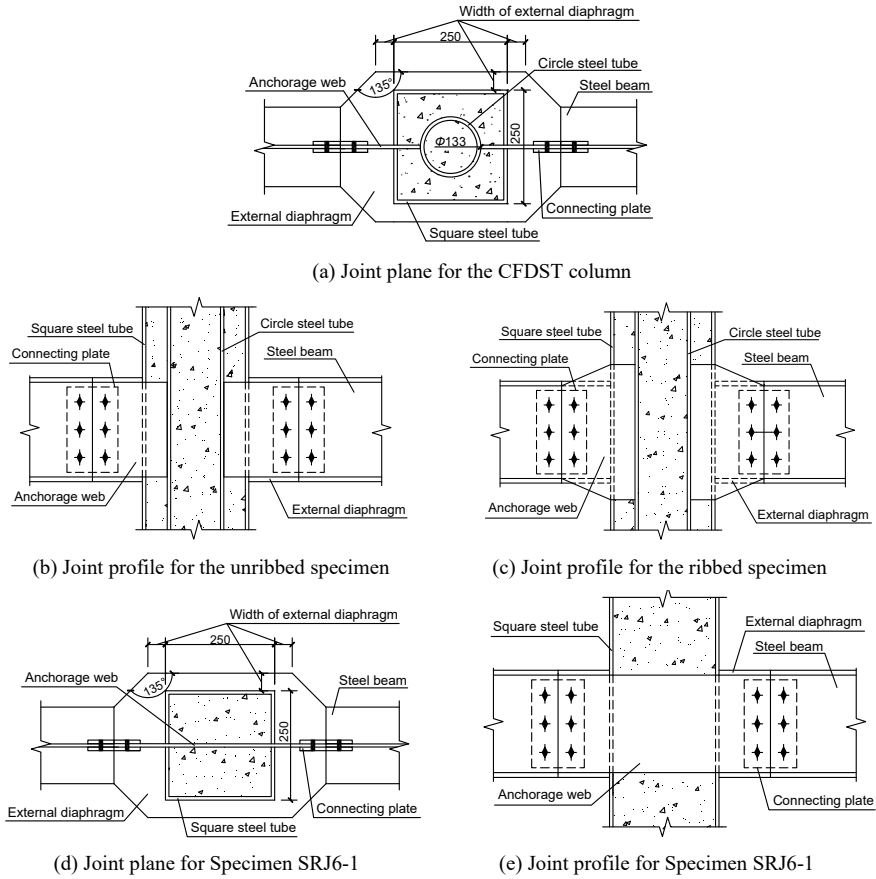


Fig. 7.1 Joint configuration (mm)

(DBJ 13-51-2003 (2003)). In this study, the width of the external diaphragm is taken as 40 mm, 60 mm and 90 mm respectively.

- *Configuration of the anchored web*: there are two types of anchored web: one with triangle ribs and the other without ribs. The rib is 51 mm high.
- *Beam-to-column bending stiffness ratio*: there are two types of joints including weak-beam joint and weak-column joint realized by two kinds of I-shaped steel beams. Beam-to-column bending stiffness ratio $k = (E_b I_b H) / (E_c I_c L)$, where $E_c I_c$ is the elastic bending stiffness of the CFDST column; $E_b I_b$ is the elastic bending stiffness of the steel beam; H is the column height; L is the beam length. $k = 0.39$ for weak-beam joint and $k = 0.71$ for weak-column joint. Besides, column-to-beam bending strength ratio is also considered as 1.15 and 1.71 for weak-beam joint and weak-column joint respectively.

Table 7.1 Geometric properties and mechanical characteristics of specimens

Specimen	$\square B \times t$	$\Phi D_i \times t$	Diaphragm width (mm)	Anchored web	Beam section	k	n
SRJ1-1	$\square 250\text{mm} \times 8\text{mm}$	$\Phi 133\text{mm} \times 6\text{mm}$	40	Unribbed	$244\text{mm} \times 175\text{mm} \times 7\text{mm} \times 11\text{mm}$	0.39	0.567
SRJ1-2	$\square 250\text{mm} \times 8\text{mm}$	$\Phi 133\text{mm} \times 6\text{mm}$	40	Ribbed	$244\text{mm} \times 175\text{mm} \times 7\text{mm} \times 11\text{mm}$	0.39	0.567
SRJ1-3	$\square 250\text{mm} \times 8\text{mm}$	$\Phi 133\text{mm} \times 6\text{mm}$	40	Unribbed	$244\text{mm} \times 175\text{mm} \times 7\text{mm} \times 11\text{mm}$	0.39	0.248
SRJ2-1	$\square 250\text{mm} \times 8\text{mm}$	$\Phi 133\text{mm} \times 6\text{mm}$	60	Unribbed	$244\text{mm} \times 175\text{mm} \times 7\text{mm} \times 11\text{mm}$	0.39	0.361
SRJ3-1	$\square 250\text{mm} \times 8\text{mm}$	$\Phi 133\text{mm} \times 6\text{mm}$	90	Unribbed	$244\text{mm} \times 175\text{mm} \times 7\text{mm} \times 11\text{mm}$	0.39	0.567
SRJ6-1	$\square 250\text{mm} \times 8\text{mm}$	–	40	Through and unribbed	$244\text{mm} \times 175\text{mm} \times 7\text{mm} \times 11\text{mm}$	0.41	0.361
SRJ4-1	$\square 250\text{mm} \times 8\text{mm}$	$\Phi 133\text{mm} \times 6\text{mm}$	40	Unribbed	$294\text{mm} \times 200\text{mm} \times 8\text{mm} \times 12\text{mm}$	0.71	0.361
SRJ4-2	$\square 250\text{mm} \times 8\text{mm}$	$\Phi 133\text{mm} \times 6\text{mm}$	40	Ribbed	$294\text{mm} \times 200\text{mm} \times 8\text{mm} \times 12\text{mm}$	0.71	0.361
SRJ5-1	$\square 250\text{mm} \times 8\text{mm}$	$\Phi 133\text{mm} \times 6\text{mm}$	90	Unribbed	$294\text{mm} \times 200\text{mm} \times 8\text{mm} \times 12\text{mm}$	0.71	0.361
SRJ5-2	$\square 250\text{mm} \times 8\text{mm}$	$\Phi 133\text{mm} \times 6\text{mm}$	90	Ribbed	$294\text{mm} \times 200\text{mm} \times 8\text{mm} \times 12\text{mm}$	0.71	0.361



Fig. 7.2 Assembling process of the anchorage web

7.1.2 Fabrication of Specimens

There are two ways to construct the external diaphragm joint specimens. One is to connect the slotted outer tubes at the connection after the anchorage web is welded with the inner tube, so this method needs to cut the outer steel tube into two pieces first, then open slots at the column end to insert anchorage webs. In this test, the beam-to-column assembly was constructed using the other method. Firstly, strain gauges were pasted on the corresponding positions to measure vertical and circumferential strains for the inner circular steel tube. Strain gauge wires were pulled out from the bottom of outer tubes. As shown in Fig. 7.2, these two steel tubes were concentrically placed and welded on a bottom plate. The outer tube was punched holes in order to weld anchorage webs on the inner tube, then external diaphragms connected with both the outer tube and the anchorage web. Concrete using coarse aggregate with small particle size was finally cast after covering the holes. Layered vibrating the concrete with the vibrator beam ensured its compactness.

7.2 Material Properties and Test Procedures

7.2.1 Material Properties

The material properties of steel are shown in Table 7.2. Three coupons of the steel tubes and sheets were tested under tension to determine their width (t), yield strength (f_{sy}), ultimate strength (f_u), and modulus of elasticity (E_s). The measured average values of the yield strength are summarized. The self-consolidating concrete was measured as 38.2 N/mm^2 in strength for three samples, and the modulus of elasticity (E_c) of the concrete was $31,300 \text{ N/mm}^2$.

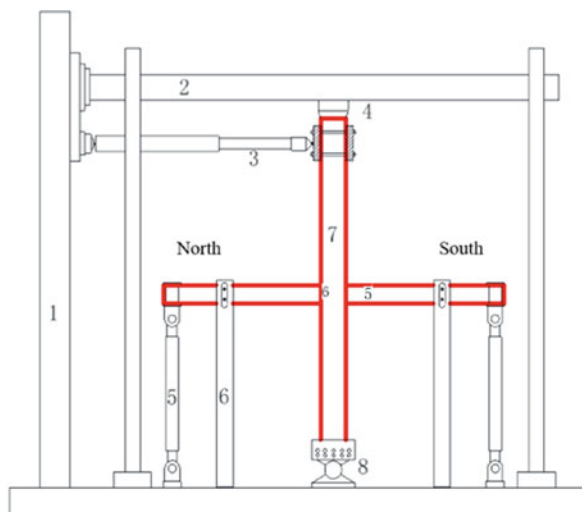
7.2.2 Test Apparatus and Procedures

In order to better simulate real earthquake action, the low-frequency cyclic loading test was conducted to load on the column top. The horizontal lateral supports were set on the column top, and the column bottom was connected with the base through a spherical hinged support, so that the stress of the specimen was close to that of the actual frame structure. The test setup included a reaction wall, a loading system and data collecting system. Figure 7.3 a provides a schematic view of the beam-to-column joint test setup. External diaphragm joint specimens were tested under a constant axial load and a cyclically increasing lateral load applied at the top of the column. As can be seen in Fig. 7.3b, the horizontal actuator is connected to the top end of the column by a rigid rod to conduct lateral loading, and a hydraulic jack with spherical hinge hanging on the top of the self-equilibrating reaction frame is used to apply a constant axial compressive force controlled by the axial compression ratio. There are pulleys inside the supporter of the hydraulic jack so the compressive force could follow the lateral displacements of the joint specimen. Articulated boundary conditions are realized through vertical supports for both north and south beam ends where the assumed inflection points of the steel beams are located. The column bottom is hinged with the foundation. Lateral braces are set to prevent out-of-plane torsion and instability of the steel beams, and there are rollers installed at the contact points to ensure beams move freely in the horizontal way.

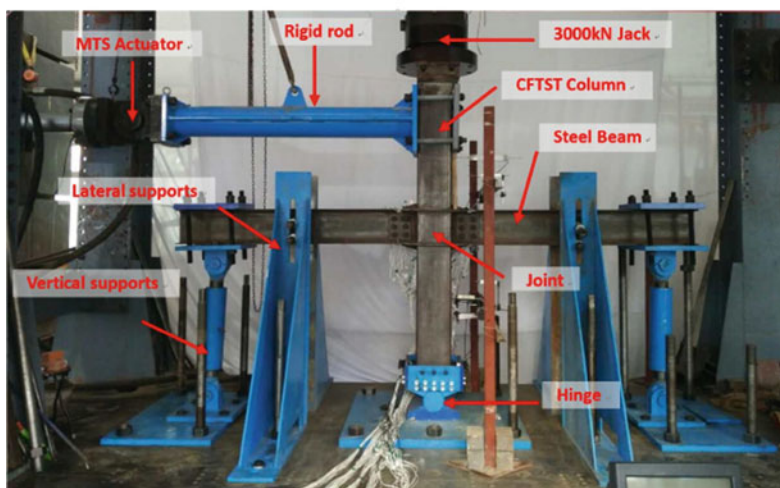
All the specimens were tested by using servo controlled hydraulic actuators, with a preprogrammed displacement history. Figure 7.4 shows the schematic diagram of the loading device. The constant axial force was applied vertically by 300t hydraulic jack. Mixed control loading mode was used in the test. The lateral loading mode was firstly adopted as controlling load generally based on the ATC-24 (1992) guidelines for cyclic testing of structural steel components. The loading process began with a load increment of 50 kN, and each loading step cycled once. When inflection points appeared in load-displacement hysteresis loops, the specimens were considered as yielded. The controlling load in inelastic cycles were then switched to lateral displacement levels of Δ_y , $1.5\Delta_y$, $2\Delta_y$, $2.5\Delta_y$, $3\Delta_y$, and two cycles were imposed at each lateral displacement levels, where Δ_y is the estimated horizontal yielding displacement corresponding to the horizontal nominal yielding load P_y .

Table 7.2 Material properties of steel

Steel plate	t (mm)	f_{sy} (N/mm ²)	f_u (N/mm ²)	E_s (N/mm ²)
Outer steel tube	8	316.2	442.95	2.15×10^5
Inner steel tube	6	362.75	440.02	2.11×10^5
External diaphragm plate	11	338.77	462.5	2.13×10^5
Steel beam (244 × 175 × 7×11)	7	327.59	460.99	2.21×10^5
Steel beam (294 × 200 × 8×12)	8	327.72	451.87	2.17×10^5
Connecting plate	6	295.31	403.89	2.01×10^5
Anchored web	8	284.51	400.79	1.98×10^5



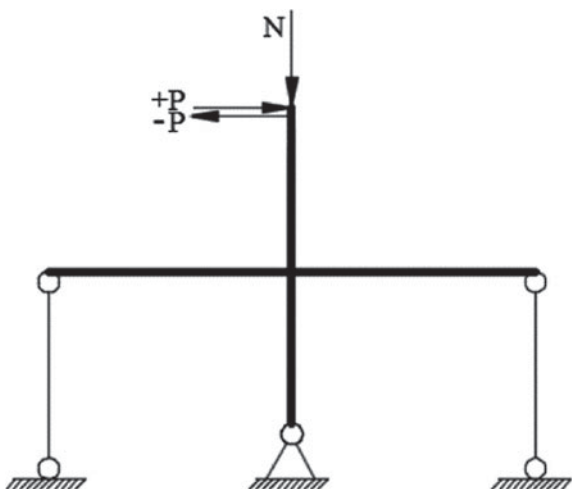
1 Reaction wall; 2 Rigid beam; 3 Electro-hydraulic servo actuator;
4 Hydraulic jack; 5 Vertical hinged support; 6 Lateral brace; 7 Specimen; 8 Roller
(a) Diagram of testing setup



(b) On-site photo

Fig. 7.3 Test setup for the external diaphragm joint

Fig. 7.4 Schematic diagram of the loading device



The load-displacement hysteresis loop was automatically collected by the electro-hydraulic servo loading system, and the data of displacement sensors and strain gauges were automatically collected by the IMP data system. To facilitate comparison, the same measurements were made for every specimen. Strain gauges were used to measure the strains in steel beam, steel tube, anchored web and external diaphragm, as shown in Fig. 7.5a. The in-plane displacements were measured by eight displacement transducers shown in Fig. 7.5b. D2 and D3 were located in the joint core zone for measuring shear deformation of the panel zone; D1, D4, D7, D8 were used to measure the absolute lateral displacement for the lower and upper column respectively, where D1, D8 were located at the point of $1\ h$ (h is the overall beam height) from the external diaphragm surface, and D4, D7 at the point of $1.5\ h$ from the surface. D5, D6 were used to measure the beam-to-column relative rotation.

7.3 Experimental Observations and Failure Modes

All the test specimens behaved in a ductile manner and the test proceeded in a smooth and controlled fashion. Firstly, the designed axial load was applied at the column top. And the beam ends moved down freely, which were later secured to the vertical supports. Then, a cyclic lateral load was applied on the column top side. The loading process of each specimen contained three stages: elastic stage, deformation developing stage and failure stage. As expected, there were two typical failure modes in general. One was beam failure mode for Specimens SRJ1-1, SRJ1-2, SRJ1-3, SRJ2-1, and SRJ3-1. The failure started from the yielding of beam flange close to the external diaphragm, and then a large deformation developed on beam flange. Wherein, steel tubes also yielded after beams' yielding, then plastic deformation

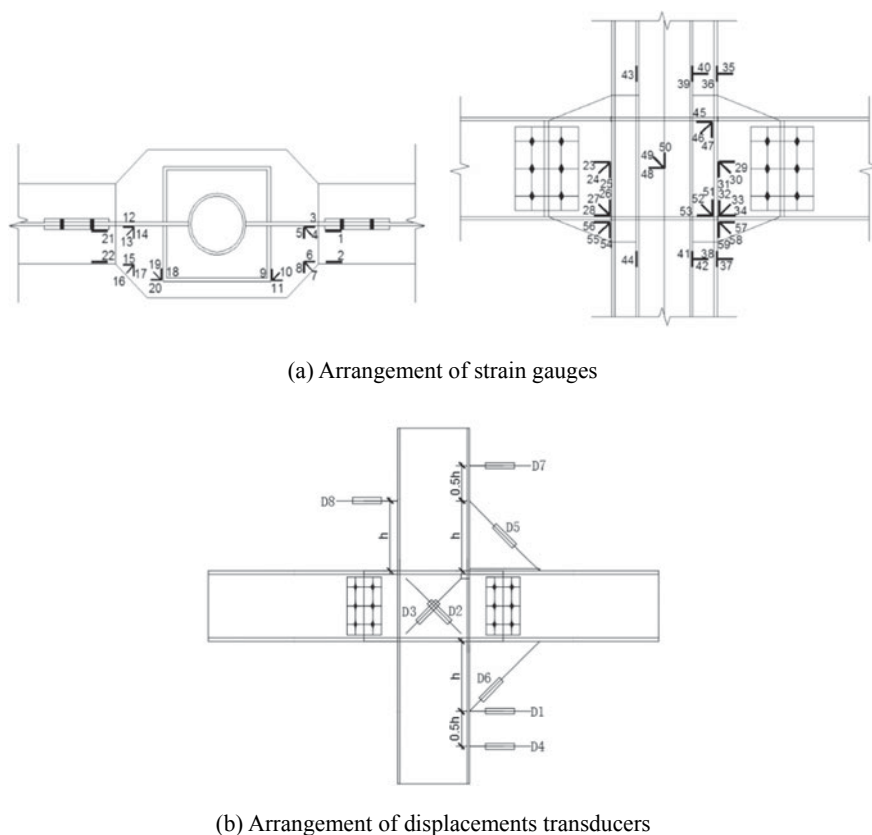


Fig. 7.5 Measurement arrangements of strains and displacements

occurred on both beam end and column end with the increasing lateral load for Specimens SRJ1-1, SRJ1-2, and SRJ3-1 under high axial compression ratio. The other was column failure mode which occurred in Specimens SRJ4-1, SRJ4-2, SRJ5-1, and SRJ5-2. The failure started from the yielding of steel tubes and flexural buckling appeared at the bottom column close to the joint core. Therefore, failure mode of the joint is mainly dependent on beam-to-column bending stiffness ratio and axial compression ratio.

7.3.1 Beam Failure Mode

Figure 7.6a ~ j reveal the failure process of the joint specimens during the test. For weak-beam joints SRJ1-1, SRJ1-2, SRJ1-3, SRJ2-1, and SRJ3-1 with small beam-to-column bending stiffness ratio, yielding occurred at the bottom beam flanges near



Fig. 7.6 Failure phenomena of the joint specimens

the external diaphragms, as shown in Fig. 7.6a. With increasing lateral load, the beam webs experienced peeling shown in Fig. 7.6b, and then the beam continued to develop a large deformation as local buckling shown in Fig. 7.6c. According to strain measurements at this stage, the strain magnitude of outer steel tube was below $1000\mu\epsilon$ for Specimens SRJ1-3 and SRJ2-1, while $1600\mu\epsilon$, $1700\mu\epsilon$, $2400\mu\epsilon$ respectively for Specimens SRJ1-1, SRJ1-2 and SRJ3-1, because the latter three specimens were tested under high axial compression ratio. Yielding occurred on both beam end and column end with the increasing load for these three specimens. Especially, strain value of outer steel tube reached nearly $3000\mu\epsilon$ for SRJ1-2 and SRJ3-1. Finally, it was evident for SRJ1-2 to be buckling at the column end (see Fig. 7.6d) under the combined action, and failure mode of SRJ3-1 is shown in Fig. 7.6e. Therefore, with the higher axial compression ratio, the column wall was easy to be bulged due to the action of axial compression and bending moment coming from the top lateral load. In spite of the weak-beam connection design, the failure mode can be changed from the beam failure to column failure with the increase of axial compression ratio, which was also described in Xu et al. (2014). It should be stressed that an ideal beam failure mode can be gotten when beam-to-column bending stiffness ratio is small, but the limit of axial compression ratio should be considered to guarantee the “strong column & weak beam” seismic design. Meanwhile, in the process of testing, the stress in the panel zone of each joint was relatively small and basically in a state of elasticity, which indicated that “strong joint” requirements were achieved by connecting CFDST columns to steel beams using external diaphragms. In contrast, many tested external diaphragm joints between CFST columns and steel beams were the weak-beam joints (Sui et al. 2019; Wang et al. 2008; Zhang et al. 2012), the steel beam exhibited obvious buckling and deformation under the common axial load level at the column top.

7.3.2 Column Failure Mode

Specimens SRJ4-1, SRJ4-2, SRJ5-1, and SRJ5-2 with large beam-to-column bending stiffness ratio failed as weak-column joints. The failure of these four joints started from the yielding of the steel tube, whilst the strain values of beam flanges were no more than $1500\mu\epsilon$, so was the shear strain in the panel zone. The steel beam and panel zone exhibited no obvious buckling and visible deformation during the test. With increasing lateral load, outer tube wall under the joint core bulged slightly (see Fig. 7.6f) and recovered with unloading. Gradually, local buckling occurred on the bottom column at the maximum lateral load, whilst there was no obvious deformation in beam flanges and webs. In the end, the strain values of beam flanges were around $2000\mu\epsilon$, getting to the yielding state, but the shear strain in panel zone was almost $10000\mu\epsilon$ already. As can be seen in Fig. 7.6g, the outer tube wall at the conjunction to external diaphragm was torn partially by shearing. The final failure modes for an unribbed specimen and a ribbed specimen are shown in Figs. 7.6h, i, respectively. Especially, the phenomena of tearing of the steel tube for ribbed specimen is shown in

Fig. 7.7 Shear strain ($\mu\epsilon$) versus lateral load (P) envelope curves of central point in panel zone

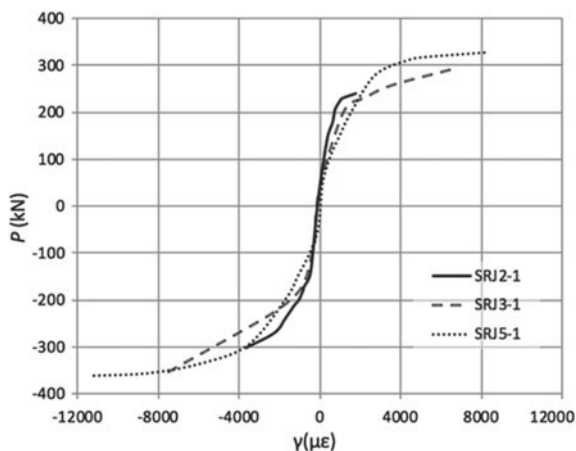
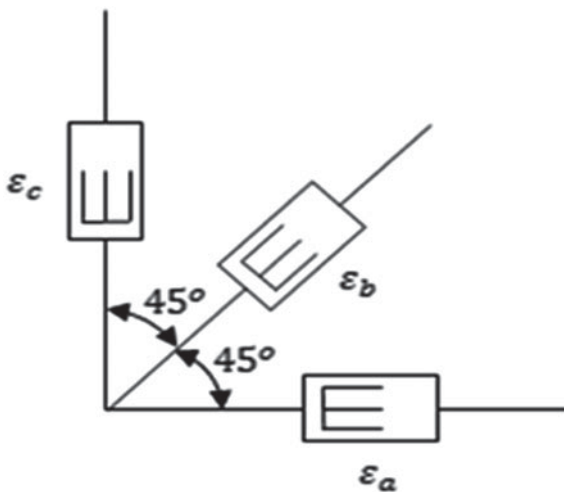


Fig. 7.6j. In general, large beam-to-column bending stiffness ratio results in column failure for external diaphragm joint in CFDST structure.

During the testing process, the shear strain in panel zone obtained by strain gauges exhibited very different in these two failure modes. Figure 7.7 shows the lateral load (P) versus shear strain (γ) skeleton curves of the bottom point (strain rosettes 51, 52, 53 in Fig. 7.5a) in panel zone before the maximum lateral load, where $\gamma = 2\epsilon_b - \epsilon_a - \epsilon_c$ obtained by the measurement of strain rosettes in this experiment. Among them, ϵ_a , ϵ_b , ϵ_c are the normal strains in horizontal, 45° and vertical directions respectively, as shown in Fig. 7.8. There are three kinds of specimens with different failure processes including SRJ2-1, SRJ3-1 and SRJ5-1. Specimen SRJ2-1 failed as beam failure mode, and the shear strain at this point increased slowly before the peak load, and it

Fig. 7.8 Strain rosette



kept small under $3900 \mu\epsilon$ all along so panel zone was in elastic stage basically. As for Specimen SRJ3-1, when the steel beam flange yielded first, the shear strain was small. Then, local buckling occurred on the CFDST column end due to the high axial compression and increasing lateral load. By this stage, the shear strain increased fast, up to $8000 \mu\epsilon$ at the peak load. Specimen SRJ5-1 failed as column failure mode, so shear strain increased rapidly before the yielding load, up to $10000 \mu\epsilon$ at the peak load. The reason is that yielded column makes the beam-to-column relative rotation increase to produce larger shear deformation in panel zone. According to another displacement measurements in panel zone (D2, D3 in Fig. 7.5b), the shear deformation in the whole panel zone can be calculated (Xiong et al. 2019), and it has almost the same increasing rate for the joint specimens under different failure modes. Because the upper or lower external diaphragm is under tension or compression to form a couple of bending moment, the force transmission results in enlarging the shear deformation in panel zone through the horizontal concentrated forces from upper and lower external diaphragms. Therefore, shear deformation should not be ignored for weak column joints. Omission of shear deformation will result in large deviation of structural analysis, which has been investigated by Du et al. (2018). For external diaphragm joints, the design should more focus on the principle of strong joints and weak members owing to the shearing capacity of the panel zone, or the specimens with weak panel zone has been verified to exhibit a shear failure in panel zone due to the local buckling (Rong et al. 2019). According to that analytical investigation, the shear capacity of external diaphragm joints is the superposition of shear capacity of core concrete and the steel tube, so for CFDST columns, the double tubes are beneficial to guarantee the shear resistance in panel zone, which can ensure the design of strong connection.

7.4 Test Results and Discussion

7.4.1 Hysteresis Loops

Load-displacement curves were automatically collected by the electro-hydraulic servo loading system. Pushing-unloading-pulling-unloading was a loading circle, and the pushing direction of the actuator was defined as positive (+), so pulling as negative (−) in the hysteresis loop of lateral load (P) versus displacement (Δ), as shown in Fig. 7.9. The negative maximum lateral load is larger than positive one because pushing load was acted before pulling in every loading cycle, and to reach the same displacement as the positive one requires a higher pulling load. All hysteresis loops present three development stages. In the first linear stage, the lateral load increases proportionally with the increase of lateral displacement at the column top, until the steel starts to yield. Then the load increases non-linearly with increasing lateral displacement up to the maximum lateral load, being characterized by spread of plasticity in material. Afterward, the hysteresis loops present a load decrease with

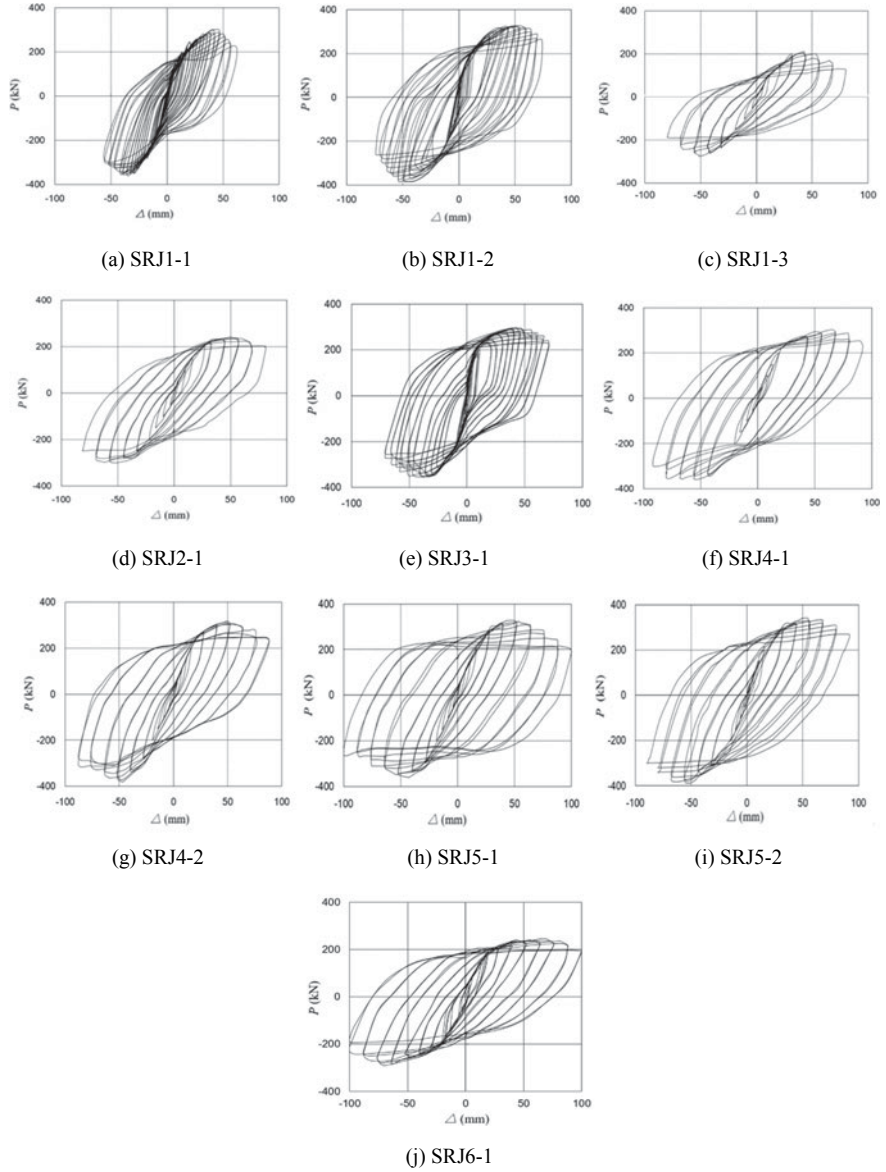


Fig. 7.9 Lateral load (P) versus displacement (Δ) hysteresis loops

increasing lateral displacement, as local buckling emerges in the steel tube or steel beam. The weak-column joint has a larger ultimate displacement and a higher bearing capacity than weak-beam joints. Since the plastic deformation occurs in column, the load-displacement hysteresis loops of weak-column joints are more stable and show a plumper shape. In contrast with the weak-beam joints, though weak-column joints have better deformation capacity, it is a risk of using weak column in industry practice (Mehrdad and Hussam 2018), so column failure should be avoided in structural seismic design.

7.4.2 Analysis of Ductility

Ductility, which can be calculated from the load-displacement curve, is defined as the ratio of ultimate to yielding displacement. The recorded skeleton curves of lateral load (P) versus lateral displacement (Δ) for the joints are described in Fig. 7.10. It can be found that the maximum load of Specimen SRJ6-1 with a square CFST column is not the smallest one. It is larger than Specimen SRJ1-3 with the CFDST column since the axial compression ratio of SRJ1-3 is smaller, and it is equivalent with Specimen SRJ2-1 since beam-to-column bending stiffness ratio of SRJ6-1 has been improved slightly. Then, the maximum load of other specimens with the CFDST column is higher than that of Specimen SRJ6-1.

The maximum load P_m obtained from the curves is shown in Table 7.3, along with the ductility coefficient ($\mu = \Delta_u/\Delta_y$, refers to Sect. 6.5.4). As can be seen from the table, the ductility coefficient lies between 2.31 and 4.51 with the average value of 2.87. In general, it is reasonable for the beam-to-column connections that the ductility coefficient is around 3.0, since it can not only fully guarantee the plastic deformation capacity of the joint without being destroyed, but also can keep the joint

Fig. 7.10 P - Δ skeleton curves of specimens

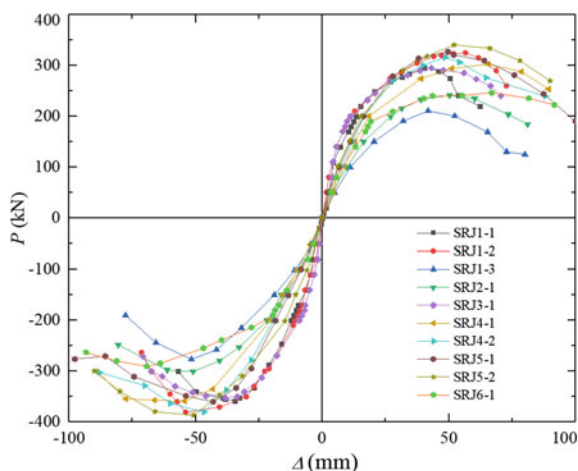


Table 7.3 Test results

Direction		Yield state		Ultimate state		Failure state		μ_u
		P_y (kN)	Δ_y (mm)	P_m (kN)	Δ_m (mm)	P_u (kN)	Δ_u (mm)	
SRJ1-1	Push	252.78	21.95	295.84	40.19	251.46	52.59	2.40
	Pull	311.42	24.18	359.13	34.51	305.26	56.18	2.32
SRJ1-2	Push	230.99	28.77	325.38	56.56	276.57	70.54	2.45
	Pull	298.12	26.96	380.91	54.00	323.77	63.77	2.36
SRJ1-3	Push	162.42	24.98	210.88	41.77	161.4	59.38	2.38
	Pull	202.03	31.28	276.75	51.7	209.74	72.27	2.31
SRJ2-1	Push	197.79	26.59	241.5	50.25	205.28	72.8	2.74
	Pull	252.2	32.65	300.94	51.01	255.8	77.8	2.38
SRJ3-1	Push	238.48	19.32	295.31	43.04	251.01	68.27	3.53
	Pull	246.75	15.01	352.75	38.27	299.84	62.71	4.51
SRJ4-1	Push	260.43	28.07	302.94	65.63	257.5	88.22	3.60
	Pull	334.28	33.02	359.06	54.36	305.2	88.17	3.23
SRJ4-2	Push	274.74	25.4	316.5	48.62	269.03	66.17	2.61
	Pull	339.28	28.32	380.54	46.71	323.46	74.33	2.62
SRJ5-1	Push	295.47	22.57	327.59	49.6	278.45	71.46	3.17
	Pull	335.45	24.84	360.34	43.15	306.29	70.08	2.82
SRJ5-2	Push	301.42	25.54	340.81	52.01	289.69	84.09	3.29
	Pull	337.72	28.4	387.16	50.59	329.1	82.73	2.91
SRJ6-1	Push	202.65	24.73	246.72	66.89	209.71	89.09	3.60
	Pull	219.49	28.47	290.69	69.36	247.09	92.06	3.23

with enough stiffness and bearing capacity (ACI 374.1-05 2014; ASCE 41-06 2007). Therefore, external diaphragm joints between CFDST columns and steel beams have good deformability and ductility. In the following, the average value of positive and negative directions are used to perform a parametric analysis.

7.4.3 Effects of Different Test Parameters

7.4.3.1 Effects of Width of External Diaphragm

Figure 7.11a, b reveal horizontal P - Δ skeleton curves of two types of joint specimens with different external diaphragm width. The only difference between the weak-beam joints SRJ1-1 and SRJ3-1 is the width of external diaphragm, which are 40 mm and 90 mm respectively, as same as the ribbed weak-column joints SRJ4-2 and SRJ5-2. As can be seen in Fig. 7.11a, the maximum lateral load and initial stiffness are almost unchanged with the increasing width for weak-beam joints, while

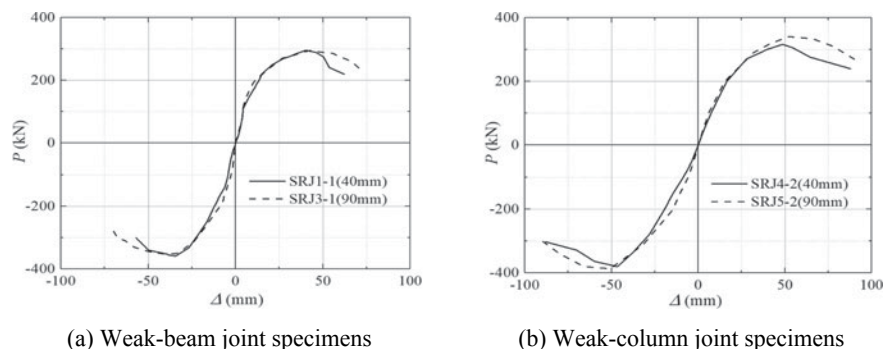
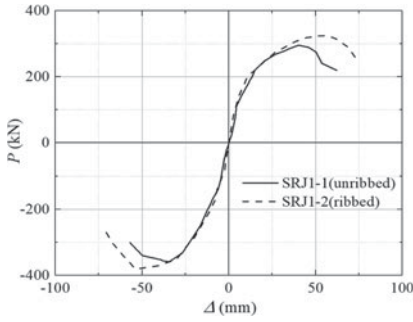


Fig. 7.11 P - Δ skeleton curves of specimens with different width of external diaphragm

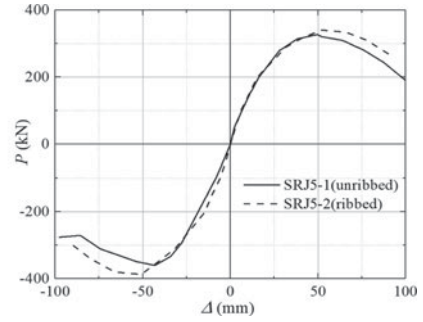
ductility significantly increases by nearly 50°. It can be realized that wide external diaphragm induces the location of plastic hinge far away from the panel zone, so joint core is still relative rigid to improve the lateral displacement at the column top. As for weak-column joints SRJ4-2 and SRJ5-2, the maximum lateral load and initial stiffness increase slightly with the increasing width, while ductility significantly rises by 18°. Although the same type of CFDSST columns failed, wider external diaphragm enhances the support restraint to provide a larger deformation space. In general, since there was no failure occurred on external diaphragms for these two types of joints, the deformation focused on the beam or column. The deformation zone of the beam would be moved away from the joint core with the increasing external diaphragm width, which results in the increase of lateral displacement at the column top. Therefore, the maximum lateral load and initial stiffness won't increase clearly with the increasing external diaphragm width, while deformability significantly increases.

7.4.3.2 Effects of Configuration of Anchored Web

Figure 7.12a, b show horizontal P - Δ skeleton curves of two types of joint specimens with different configurations of anchored web. The ibbed anchored web has an influential effect on the ultimate strength and ductility for both weak-beam joint and weak-column joint, as shown in Fig. 7.12a, b respectively. The aximum bearing capacity of ribbed specimens SRJ1-2 and SRJ5-2 increases by nearly 10% compared with unribbed specimens SRJ1-1 and SRJ5-1 respectively, and the ductility has been slightly improved. The reason is that ribbed anchored web can enhance the beam-to-column conjunction and restrict beam-to-column relative rotation for both two types of joints under different failure modes. Then yielding region on beam or column moves far away from the joint core in the later lateral loading process, so destruction occurred at larger lateral displacement for ribbed specimens. But there is another



(a) Weak-beam joint specimens



(b) Weak-column joint specimens

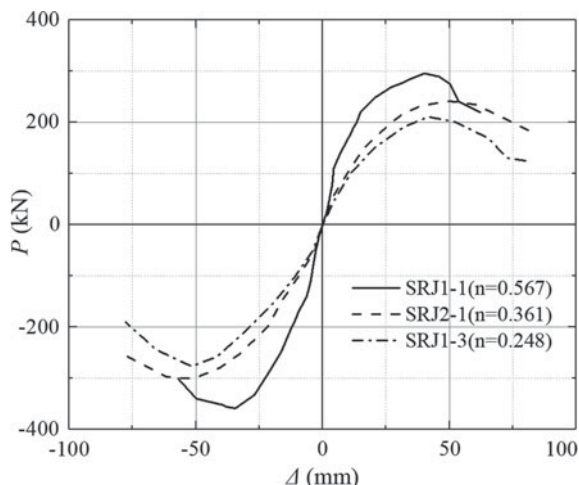
Fig. 7.12 P - Δ skeleton curves of the specimens with different anchored web configurations

problem here needs to note that the ribbed specimen will have strong stress concentration at the conjunction between ribs and outer steel tubes to result in tearing of the steel.

7.4.3.3 Effects of Axial Load Level

In the test, a loading system at the column top was adopted, so the influence of N - Δ effect was evident. Between Specimens SRJ1-1 and SRJ1-3, the only different parameter is n from 0.567 to 0.248, but for SRJ2-1 n is 0.361 and the diaphragm width is 60 mm, different from the former two specimens (40 mm). According to the aforementioned investigations, wider external diaphragm will not obviously increase maximum lateral load and initial stiffness, but it just improves the ductility significantly. Consequently, it is reasonable to put Specimens SRJ1-1, SRJ1-3, and SRJ2-1 together to analyze the influence of n . Figure 7.13 shows horizontal P - Δ skeleton curves of three joint specimens with different axial compression ratio (n). As can be seen, the initial stiffness of the joints increases evidently with the increasing n , because the lateral load needs to increase to reach the same displacement when the higher compression acts at the column top, and it may also attributes to the friction increase from the loading device. Similarly, the maximum bearing capacity increases significantly as well. Maximum bearing capacity of SRJ1-3 decreases by about 30% than SRJ1-1. Additionally, the lateral load goes down dramatically after peak value owing to the larger axial compressive force. Therefore, it is better to control the axial compression ratio to avoid bending failure of the CFDST column in engineering applications.

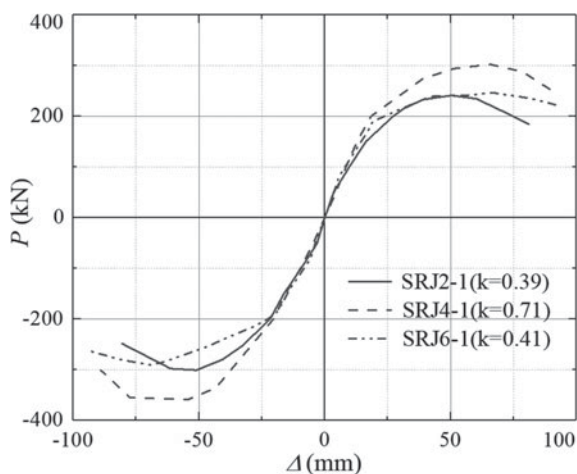
Fig. 7.13 P - Δ skeleton curves of the specimens with different axial compression ratios



7.4.3.4 Effects of Beam-to-Column Bending Stiffness Ratio

Specimen SRJ2-1 with the diaphragm width 60 mm is regarded as the same maximum bearing capacity as the joint with the diaphragm width 40 mm according to the foregoing statement, so SRJ2-1 and SRJ4-1 for CFDST columns and Specimen SRJ6-1 for square CFST column are put together to analyze the influence of different beam-to-column bending stiffness ratio (k) using P - Δ skeleton curves shown in Fig. 7.14. It can be concluded that the maximum lateral load of weak-column joint SRJ4-1 increases 22.4% more than weak-beam joint SRJ2-1 due to the different failure modes. As described before, k influences the failure mode, so weak-column joint and weak-beam joint exhibit different mechanical behaviors. For example, although the

Fig. 7.14 P - Δ skeleton curves of the specimens with different beam-to-column bending stiffness ratios



initial stiffness increases little, later flexural stiffness improves greatly for the weak-column joint with large k since the increase of k changes the distribution of internal forces between the beam and the column, then k also influences the bearing capacity of the joint. While for different column types, there was large bending deformation in the same type of steel beam for both of Specimens SRJ2-1 and SRJ6-1. Although SRJ6-1 has the column of a square CFST, its bearing capacity is equivalent with Specimen SRJ2-1 since the beam-to-column bending stiffness ratio of SRJ6-1 has been improved slightly.

7.4.4 Stiffness Degradation

For various reasons, all the specimens suffered the stiffness degradation with damage accumulation during cyclic loading. The stiffness is estimated by computing slope of the line joining the peak load and zero load at half cycle of each displacement amplitude. The average stiffness of each displacement amplitude can reflect stiffness characteristics with the increase of the displacement. The index of cyclic stiffness (K_i) is used to describe the degradation of the stiffness, which is defined by Nie et al. (2008). Take the average stiffness under same displacement level: $K_i = \sum_{i=1}^n P_j^i / \sum_{i=1}^n \Delta_j^i$, in which, K_i is loop stiffness; P_j^i is the peak load value of the i th cycle when the displacement control is j ; Δ_j^i is the column end displacement of the i th cycle when the displacement control is j ; n is the loading cycle. Figure 7.15 shows the stiffness degradation of the specimens versus the applied displacement at column top. The curves about pushing and pulling loading are basically symmetry in the positive and negative directions. It can be seen that the stiffness degradations of all joint specimens are obvious, continuous, uniform and stable during the entire loading process. The initial stiffness of pushing load (positive direction) is lower than the pulling load

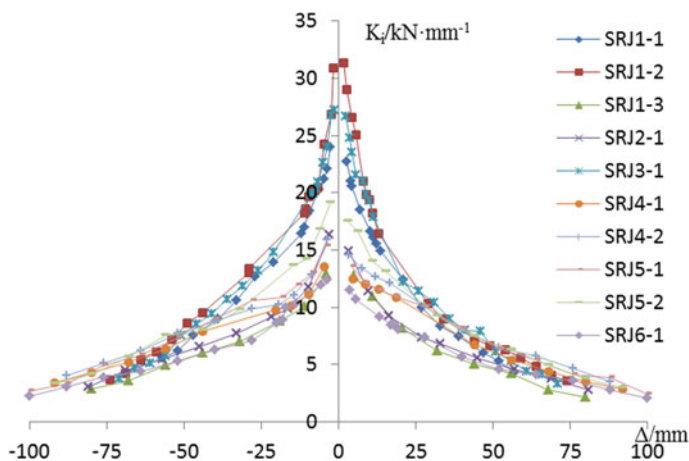


Fig. 7.15 Stiffness degradation of specimens

(negative direction) because all the column tops were pulled after pushing to form a loading circle at each loading step. On the other hand, the ribbed joints with wide external diaphragms show a higher loop stiffness. The axial compression ratio has an obvious function on the stiffness degradation. The initial loop stiffness of Specimens SRJ1-1, SRJ1-2 and SRJ3-1 is larger, while the cumulative damage increased after yielding of the specimen because the N - Δ effect is remarkable under high axial compression ratio, so stiffness degrades sharply with increasing lateral displacement.

7.4.5 Energy Dissipation

The relative beam-to-column rotation (θ) was taken by the displacement transducers in the test (see Fig. 7.5). The lateral force was applied on the side of the column top, so bending moment (M) at the beam end can be calculated using the equilibrium equation. After the emergence of plastic hinges, the deformation of the specimen depends mainly on plastic rotation of the plastic hinge zone. Each of the tests culminated in the formation and subsequent propagation of fatigue cracks. Therefore, the hysteresis loops in spindle form, show good energy dissipation capacity of this type of joint. With the increase of displacement, the energy dissipation increased obviously for all the joint specimens in the post yield stage.

In this work, the energy dissipation capacity can be evaluated by the equivalent viscous damping coefficient h_e , which is defined as $h_e = E/2\pi$ (Bai et al. 2008), herein E is the energy dissipation coefficient of every hysteresis loop described in Fig. 7.16. $E = S_{BFDE}/(S_{OAB} + S_{OCD})$, where S_{BFDE} represents the area in loops $BFDE$; and S_{OAB} and S_{OCD} represent shaded areas within the triangles OAB and OCD , respectively. Based on the M - θ hysteresis loop of the joint specimen, the energy dissipated in per loading circle can be quantitatively obtained. Figure 7.17 provides the equivalent viscous damping coefficient (h_e) of the joint specimens after the yielding point. The equivalent viscous damping coefficients for all specimens increase quickly with the increasing loading displacement. Since the area of the hysteresis loop indicates

Fig. 7.16 Idealized P - Δ hysteresis relationship

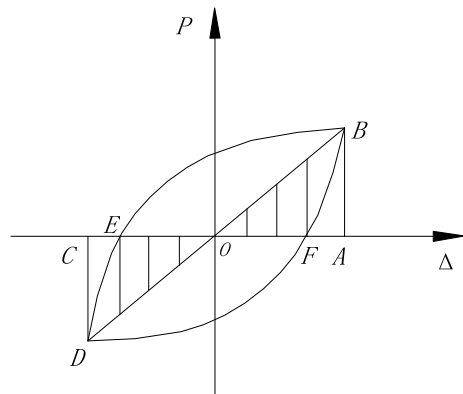
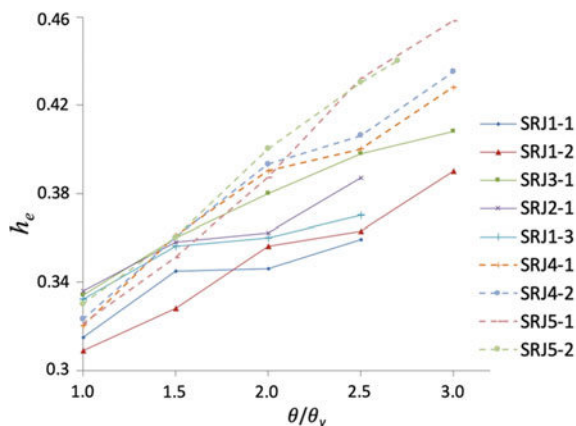


Fig. 7.17 Equivalent viscous hysteretic damping coefficient



the energy dissipation in each cycle, these plots give a visual representation of the energy dissipation. As depicted in Fig. 7.17, at the yielding points, h_e belongs to the small value, but h_e increases after yielding and h_e is around 0.34 for all specimens at peak load. Then h_e changes with different parameters. In combination with the test phenomena, the conclusions are: (1) Weak-column joints SRJ4-1, SRJ4-2, SRJ5-1 and SRJ5-2 have a better energy dissipation capacity than weak-beam joints SRJ1-1, SRJ1-2, SRJ1-3, SRJ2-1, and SRJ3-1, because the deformation under the column failure mode is larger. For example, the shear deformation in panel zone as described in Sect. 7.4.2 indicates the component with larger deformation can help to absorb more energy; (2) The joints with wide external diaphragm or triangle ribs such as SRJ3-1, SRJ4-2, and SRJ5-2 have better energy absorption, because the improved connection stiffness can make the yielding zone farther away from the joint core, then the deformability increases; (3) The joints with higher axial compression ratio, such as SRJ1-1, and SRJ1-2, have a worse energy dissipation ability, because the ductility decreases and there is a relatively sharp decline in hysteresis loops.

7.5 Data Analysis Based on DSCM

7.5.1 DSCM and Measurement Setup

The digital speckle correlation method (DSCM) is a new non-contact optical technique, which can be used to measure full strain field in the panel zone and the beam-to-column relative rotation in the cyclic loading test for new external diaphragm joints of CFDST structures. DSCM can obtain the displacement fields on a specimen's surface by comparing local correlation of two images before and after deformation. In contrast with traditional measurement methods, DSCM has the advantages of low-cost, high precision, non-destructive contact, and a relatively easy procedure, thus it

has been utilized to analyze various problems. According to the principles and characteristics of DSCM technology, researchers (Song et al. 2015; Sun et al. 2008) have extended their applications to many fields such as mechanics of machinery, wood mechanics, biomechanics, rock mechanics, fracture mechanics, and so on. Lee et al. (2010) used it in an experimental study to measure and observe the full strain field of the joint, while Stephen et al. (2008) used it to observe the interfacial shear stress of RC beams strengthened with FRP. Some researchers have also used this method in the experimental investigation of composite structures (Lagattu et al. 2005; Hufner and Accprsi 2009).

Conventional strain gauges only can offer measured values at single points, but the principal advantage of the DSCM method used in this chapter is that displacement measurement and strain measurement are continuous over the total joint zone. The cameras to collect measuring points were placed at two meters in front of the specimen. Two CCD cameras were used to collect speckle areas from beam-to-column corner and joint panel zone respectively, as shown in Fig. 7.19b. The image sampling frequency of camera was 1 Hz. The collected images were stored in the bmp format with a maximum resolution of 1600×1200 by a laptop. Some random speckle patterns were painted on the surface of the specimen, and an original image of undeformed specimen was acquired before the test. During the test, additional images were acquired at each step.

The experimental data collections contained the beam-to-column relative rotation and shear deformation in the panel zone for only seven specimens. Camera-1 was used to measure shear deformation in the panel zone, and measuring points and speckle arrangements are shown in Fig. 7.18. Camera-2 was used to measure the beam-to-column relative rotation, and its lens was to focus on the beam-to-column corner. The relative rotation was obtained by a triangle with three measuring points at the beam-to-column junction, beam flange edge, and column edge respectively, as shown in Fig. 7.19. The measuring points on the column edge and the beam edge

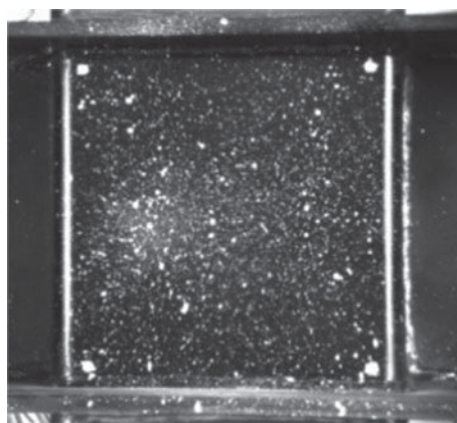
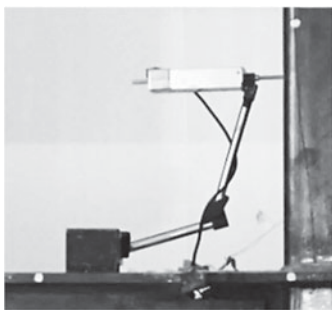


Fig. 7.18 Measuring points and speckle arrangements in panel zone



(a) Measuring points at upper north corner



(b) Measuring points at lower north corner

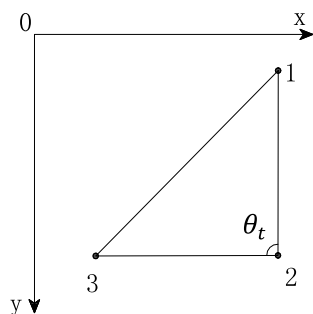
Fig. 7.19 Measuring points on the corners

were a beam height from the conjunction point, which was located on projection position of external diaphragm edge at the beam-to-column intersection. There were two kinds of rotation data collections of the upper north corner (Fig. 7.19a) and the lower north corner (Fig. 7.19b). The upper north corner was carried out for Specimens SRJ1-1, SRJ1-2, and SRJ6-1, and the lower north corner for SRJ1-3, SRJ2-1, SRJ4-2, and SRJ5-1.

7.5.2 Data Processing

7.5.2.1 Calculation of Beam-to-Column Rotation

MATLAB program was used to recognize each image automatically. Since each image has the same size, i.e., the number of speckles per image was the same, then a coordinate system was established, as shown in Fig. 7.20. Each speckle was a point in the coordinate system, and it was considered to have a displacement and a rotation in the plane, so speckles can directly calculate the angle displacement change and the shear deformation according to the coordinate value of the measuring points. Take the

Fig. 7.20 Coordinate system in MATLAB

upper north corner as an example, the coordinate system of three measuring points (x_1, y_1) , (x_2, y_2) , (x_3, y_3) is establish. According to the triangular cosine formula, the actual angle values of the beam-to-column at each moment can be obtained. Then the beam-to-column rotation θ is

$$\theta = \theta_t - \theta_{t_0} \quad (7.1)$$

where θ_t is the actual angle value at moment t , as shown in Fig. 7.20. θ_{t_0} is the actual angle value at the initial moment t_0 .

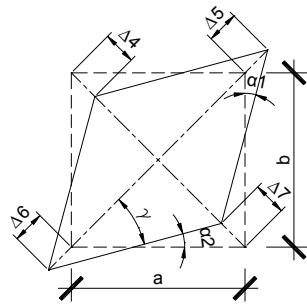
7.5.2.2 Calculation of Shear Deformation in Panel Zone

Under the action of the reciprocating load, the whole joint core has the tendency of changing from an original rectangle to a diamond shape as shown in Fig. 7.21, and the direction of the diamond changed alternately with pushing and pulling a lateral load. The images data collected by DSCM were identified by MATLAB using the coordinates (x_4, y_4) , (x_5, y_5) , (x_6, y_6) , (x_7, y_7) of four measuring points (see Fig. 7.18). The formula in Tang (1989) was first put forward to calculate the shear angle deformation (γ) in the whole panel zone as shown in follows.

$$\gamma = \alpha_1 + \alpha_2 = \frac{1}{2}(\Delta_4 + \Delta_5 + \Delta_6 + \Delta_7) \frac{\sqrt{a^2 + b^2}}{ab} \quad (7.2)$$

where $\Delta_i = \sqrt{(x_i - x_{i0})^2 + (y_i - y_{i0})^2}$, $i = 4, 5, 6, 7$. $(x_{i0} - y_{i0})$ is the initial coordinate of the measuring point, and Δ_4 , Δ_5 , Δ_6 , Δ_7 are the displacements of the angular points as shown in Fig. 7.21.

Fig. 7.21 Shear deformation of the whole panel zone



7.5.2.3 Determination of Bending Moment and Shear Force in Panel Zone

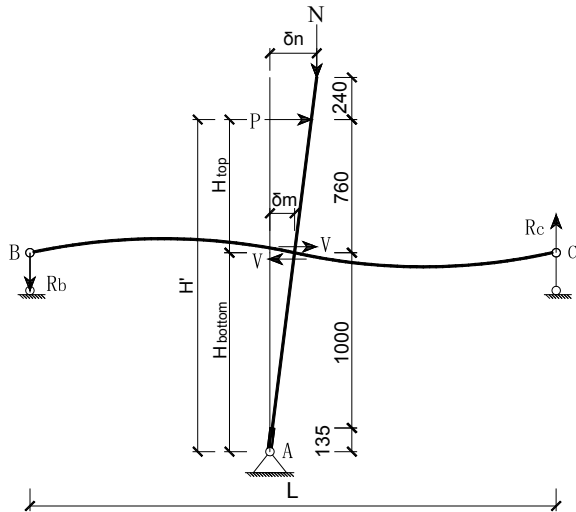
Though the test specimen was designed as symmetrical beam-to-column assemble structure, the actual lateral load was exerted by two plates clamping the top column end, so the equivalent acting point was away from the column top. Meanwhile, the distance between the column base and the center of the bottom hinge was 135 mm, then calculation length of the top column was less than that of the bottom column. The simplified calculation diagram of the specimen is shown in Fig. 7.22. Take the top column and bottom column as an isolated body respectively, and draw free body diagram, then the calculation formula of bending moment is

$$M_j = \begin{cases} P \cdot H'_{top} + N \cdot (\delta_n - \delta_m) & (\text{For upper north corner}) \\ P \cdot H'_{bottom} + N \cdot \delta_m & (\text{For lower north corner}) \end{cases} \quad (7.3)$$

where P is lateral load at the column top. N is the vertical axial load at the column top; H'_{top} is the distance between the lateral load acting point and the top surface of the steel beam; H'_{bottom} is the distance between the bottom hinge center and the bottom surface of the steel beam. δ_n is the lateral displacement of column top. δ_m is the lateral displacement of joint central point. Due to the lateral load at the top of column, the panel zone bears the horizontal shear force to produce corresponding shear deformation. As can be seen in Fig. 7.22, the average shear force in panel zone V is calculated by

$$V = \frac{2(PH' + N\delta_n)L_b}{hL} - P \quad (7.4)$$

Fig. 7.22 Schematic of loading forces acting on the beam-to-column joint



where H' is the distance between the lateral load acting point and the hinge center. L is the distance between the two constraint supports. L_b is the distance from one constraint support to column outside surface. h is the height of the steel beam.

7.5.2.4 Hysteresis Loops of Moment-Rotation ($M-\theta$)

According to the beam-to-column relative rotation θ (Eq. 7.1) and the column end bending moment M (Eq. 7.3), $M-\theta$ hysteresis loop is obtained during all the loading process, as shown in Fig. 7.23. As can be seen in Fig. 7.23, $M-\theta$ hysteresis loop is full and in a spindle shape; with the increase of load the hysteresis loop is more and more plump with no pinch, showing good energy dissipation. In the initial loading process, the curves show linear change for the specimen is in the elastic stage. When the specimen enters into the yielding state, θ begins to increase fast, the slope of the hysteresis loop decreases gradually, so stiffness degrades sharply during the later loading process.

By comparing Fig. 7.23a, b, it can be concluded that the relative rotation θ of upper north corner is much smaller than that of lower north corner. Because in Eq. 7.3 the bending moment of bottom column end is about 1.5 times that of the top column, and the ultimate rotation of the lower north corner is about two times over that of upper north corner. As can be seen in Fig. 7.23a, the $M-\theta$ curve shows a descent during the later phase especially for Specimens SRJ1-1 and SRJ1-2. The main reason is that stiffness's redistribution due to the material's yielding resulted in reduction of the top bending moment, while the deformation recovery was lagging behind.

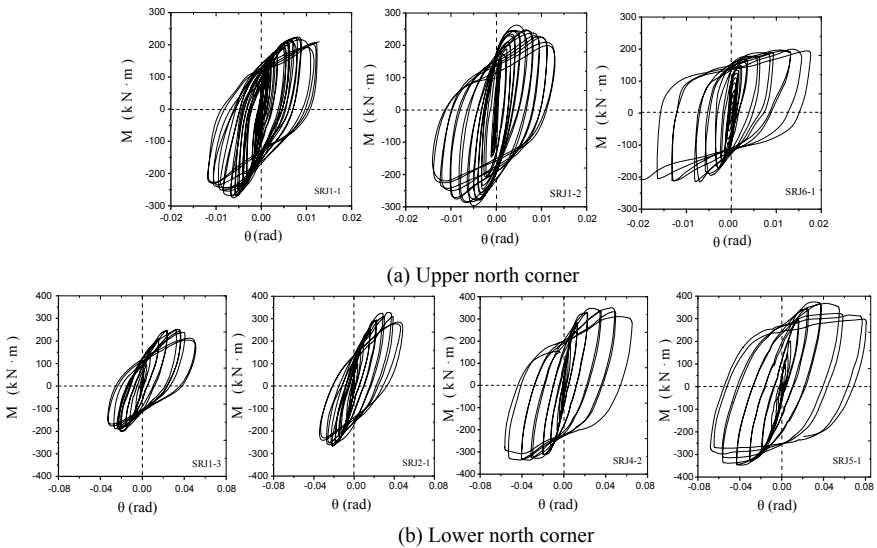


Fig. 7.23 Hysteresis loops of moment-rotation

So, the measured $M-\theta$ curves of upper north corner cannot reflect the mechanical behavior well after yielding state of the joint. Then, the remaining specimens were changed to measure the lower north corners, and better results have been achieved. By analyzing the four hysteresis loops in Fig. 7.23b, the hysteresis loops of the joints SRJ4-2 and SRJ5-1 with larger beam-to-column bending stiffness ratio are obviously plumper than the joints SRJ1-3 and SRJ2-1 with smaller one, and the rotation is larger inevitably. So with the increase of beam-to-column bending stiffness ratio, the energy dissipation capability was improved.

7.5.3 Joint Stiffness Analysis

The skeleton curves of $M-\theta$ of the upper north and lower north corners can be obtained respectively, as shown in Fig. 7.24. It can be observed that all curves have obvious yield points, and Specimens SRJ4-2 and SRJ5-1 with large beam-to-column bending stiffness ratio have the higher maximum lateral load and exhibit better ductility and plasticity. By comparison of the four joints tested with the lower north rotation, Specimens SRJ2-1, SRJ5-1 with wider external diaphragm present the better ductility. That is because the wide external diaphragm can make steel beam yielding away from the joint core, so the joint core is still relatively rigid to improve the deformation ability.

The classification of joint stiffness can be determined by the initial stiffness through the relationship between the moment and the relative beam-to-column rotation. According to the definition of the rotational stiffness of a joint in Eurocode 3: BS EN 1993-1-8 (2005), joints are classified by their initial rotational stiffness into rigid, semi-rigid or pinned as shown in Table 7.4. Herein, $S_{j,ini}$ is the initial elastic rotational stiffness, EI_b/L_b is the line stiffness of the beam. The non-sway structure specified here refers to a strong support system to strengthen it, so 80% of lateral

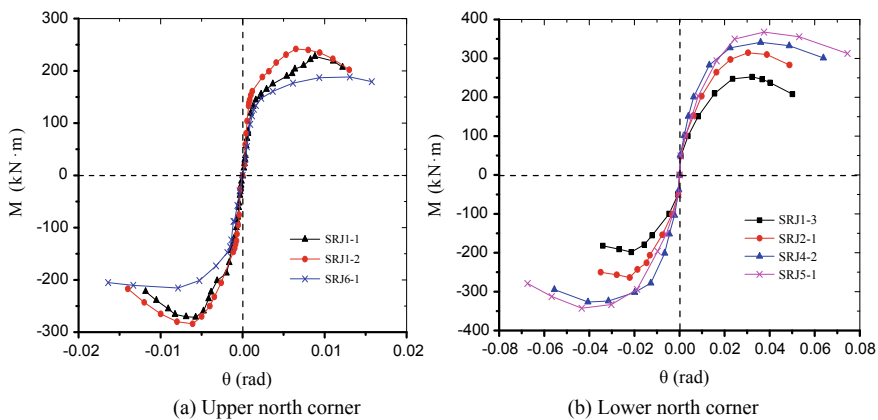


Fig. 7.24 Skelton curves of moment-rotation

Table 7.4 Classification of joint stiffness

Stiffness classification	Non-sway structure	Sway structure
Rigid	$S_{j,ini} \geq 8EI_b/L_b$	$S_{j,ini} \geq 25EI_b/L_b$
Semi-rigid	$8EI_b/L_b > S_{j,ini} > 0.5EI_b/L_b$	$25EI_b/L_b > S_{j,ini} > 0.5EI_b/L_b$
Hinge	$S_{j,ini} \leq 0.5EI_b/L_b$	$S_{j,ini} \leq 0.5EI_b/L_b$

displacement can be reduced at least. The external diaphragm joint herein belongs to the sway structure. The initial rotational stiffness of each joint (Bai et al. 2008) in this research is obtained by calculating the slope of the linear part of the moment-rotation curve as following.

$$S_{j,ini} = \frac{M_e}{\theta_e} \quad (7.5)$$

where M_e is the elastic bending capacity in the elastic stage of $M-\theta$ curve. θ_e is the beam-to-column relative rotation corresponding to M_e . Table 7.5 shows the computed initial rotational stiffness of these seven joints, and the initial rotational stiffness of each specimen is less than $25EI_b/L_b$, but it is close to $25EI_b/L_b$ and much larger than $0.5EI_b/L_b$ especially for the ribbed specimens. Therefore, though the external diaphragm joint can be classified as a semi-rigid joint in a sway structure, it is still a similar rigid connection. Researches (Qiao and Wang 2005; Gizejowski et al. 2006; Park and Wang 2011) concluded that most of the joints in practical engineering were semi-rigid and they exhibited better seismic performance, deformation and energy dissipation. Especially, the connection between a steel beam and a CFST column was generally taken as a semi-rigid connection, and it was between the extreme two cases of the hinge and rigid joints (Reyes-Salazar and Haldar 1999; Sophianopoulou 2003). The relative beam-to-column rotation was measured at the location out of the external diaphragm width from the joint core in this study, so the effect of rotation length could not be ignored either. Moreover, the shear deformation in the panel zone was very small as described in the aftermentioned section, indicating that the

Table 7.5 Initial rotational stiffness of joints

Specimen no.	Positive initial stiffness (kN·m)	Negative initial stiffness (kN·m)	$25EI_b/L_b$ (kN·m)
SRJ1-1	85484	83417	93963
SRJ1-2	90625	91796	93963
SRJ1-3	79257	78204	93963
SRJ2-1	89452	90476	93963
SRJ6-1	80488	80766	93963
SRJ4-2	148709	134272	170782
SRJ5-1	105537	110118	170782

external diaphragm joint can meet the earthquake-resistant design requirement of “strong joint & weak member”.

As shown in Fig. 7.23 and Table 7.5, the initial stiffness of specimens with the ribbed web stiffened (SRJ1-2 and SRJ4-2) is greater than those corresponding unribbed specimens. This suggests that the initial rotation stiffness of the joint will be improved by adding stiffeners to the anchor web, and the bending resistance has also increased by 10%. The comparison of SRJ1-3 and SRJ1-1 indicates that the decrease of initial stiffness may be attributed to the level of axial load on the CFDST column because the beam-to-column relative rotation capacity can be limited with the increase of axial compression ratio. Larger beam-to-column bending stiffness ratio leads to higher initial rotational stiffness of Specimens SRJ4-2 and SRJ5-1, as shown in Fig. 7.23b; meanwhile, the ultimate bearing capacity is increased significantly with the increase of the beam-to-column bending stiffness ratio. Through the comparison between SRJ6-1 and SRJ1-3, as shown in Table 7.5, the initial stiffness of the two specimens is basically equal though the axial compression ratio of SRJ6-1 is slightly higher than SRJ1-3. This means the anchored web welded to the inner tube for CFDST column has the same function as penetrating anchored web for ordinary CFST column.

7.5.4 Relationship of Shear Force-Deformation in Panel Zone (V - γ)

Shear deformation of panel zone is the primary mode of deformation in beam-to-column joints, and it provides a measure of the level of damage suffered by the joint. Different researchers use different approaches to measure shear deformation of the joint region. DSCM can provide an accurate prediction of shear deformation of the whole panel zone. The deformation characteristic in the panel zone was different for various joint specimens under beam failure mode and column failure mode, so the entire process of shear deformation was analyzed in the experiment. According to the aforementioned Eqs. 7.2 and 7.4, V - γ hysteresis loops of the whole panel zone are obtained as shown in Fig. 7.25. The hysteresis loop of SRJ1-2 is not given here due to the failure of CCD camera. It can be seen from Fig. 7.25 that the V - γ hysteresis loops are all homogeneous and fusiform, indicating that panel zone can have some certain energy dissipation capacity by shear deformation. The value of shear deformation in panel zone is relatively small during the whole loading process because the deformation mainly focuses on the beam or column. For Specimens SRJ1-1, SRJ1-3, and SRJ2-1 with the weak beam configuration, shear deformation γ is no more than 0.01 rad before the peak load as shown in Fig. 7.25a-c. It is assumed in this research that the specimens under beam failure mode did not enter shear yielding state in panel zone, so the shear deformation can recover back after the maximum lateral load. Among these four specimens, the measured maximum shear deformation of SRJ6-1 is the largest because only outer steel tube bears shearing in

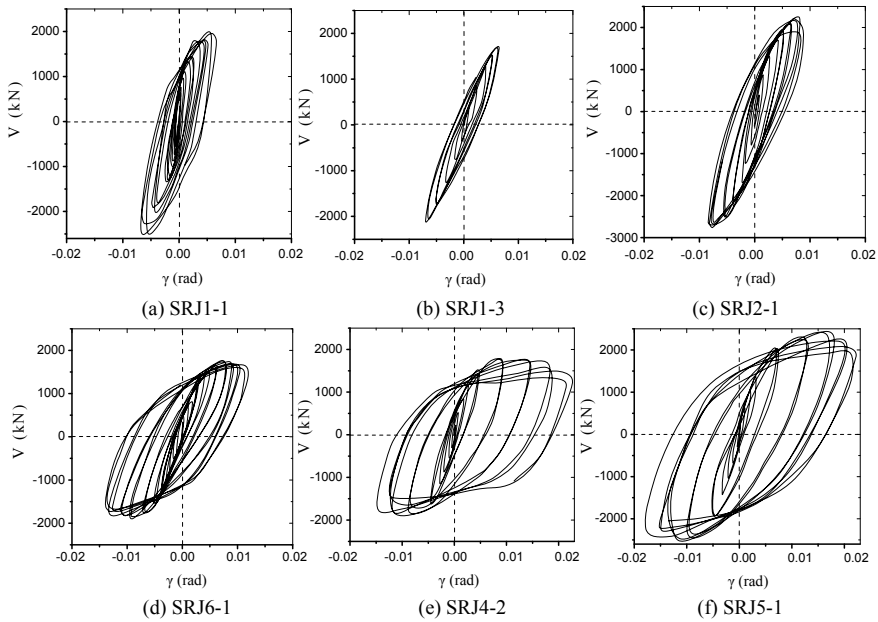
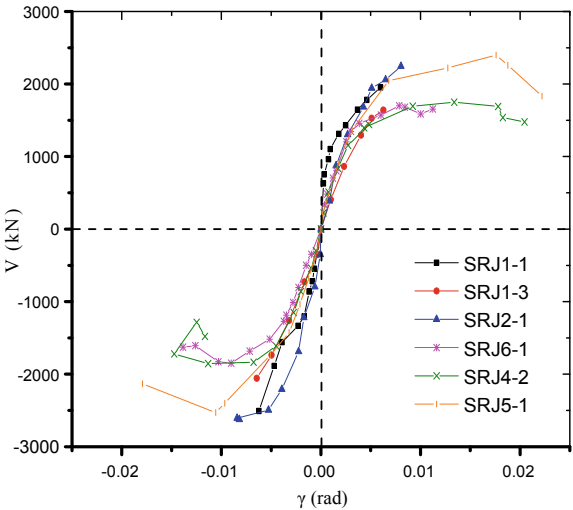


Fig. 7.25 Hysteresis loops of V - γ in panel zone

the panel zone. However, double tubes in the CFDST column decreased the shear deformation a lot. Figure 7.25e, f show V - γ hysteresis loops for Specimens SRJ4-2 and SRJ5-1 with weak-column configuration. Due to the increase of beam-to-column bending stiffness ratio (k), the predicted results are up to a larger shear deformation as 0.023 rad. This indicates that the panel zone entered into a preliminary elastic-plastic state for specimens under column failure mode in the test.

The V - γ skeleton curves are shown in Fig. 7.26. It can be observed that the initial shear stiffness of all CFDST joints except SRJ1-3 is larger than the joint SRJ6-1 of square CFST. It is verified that the inner circle steel tube in the CFDST column can improve the shear stiffness. However, the axial load level is the main influential parameter on shear stiffness (Liu and Jia 2018), so SRJ1-3 with the minimum axial load level in the test has the smallest shear stiffness and Specimen SRJ1-1 has the largest initial shear stiffness. Specimens SRJ2-1 and SRJ5-1 with wider external diaphragm also have larger initial shear stiffness because the wide external diaphragm can improve the ability to limit the tendency of mutual dislocation in the panel zone as shown in Fig. 7.21. Most notably, beam-to-column bending stiffness ratio (k) influences the shear performance in panel zone of the external diaphragm joint evidently. The earlier yielding of the specimens with weak-beam configuration led to insufficient development of shear deformation, and the shear deformation of SRJ4-2 and SRJ5-1 entered into the preliminary elastic-plastic state due to the increase of beam height. This is because the stronger beam can limit the beam-to-column relative rotation resulting in a stress improvement in panel zone, but the final failure

Fig. 7.26 Skeleton curves of V - γ in panel zone



mode was still the type of column failure, not a shear failure in panel zone. It can be concluded that the shear deformation of SRJ6-1 may be affected by the improvement of k because there was no circular tube inside. Therefore, the increase of k not only improves the bending resistance and energy dissipation capacity of joints but also develops the shear deformation in the panel zone greatly.

Table 7.6 presents the values of beam-to-column rotation θ of the lower north corner and the corresponding shear deformation γ in the panel zone at the maximum lateral load in the test. It can be found that Specimens SRJ4-2 and SRJ5-1, which failed by column failure, has a larger value of the ratio γ/θ than that of the other two specimens with beam failure. Overall, the average value of the shear deformation γ accounts for 35% of the rotation θ , so γ has a significant contribution to θ . This is attributed to the enhancement function of the upper and bottom external diaphragms under the action of the overturning moment. The pulling or pushing force transferred from beam flanges through the external diaphragm produced large shear deformation in the panel zone. In addition, the loading method at the column top adopted in the experiment and the secondary effect of the axial force could also increase the shear deformation in the panel zone. Therefore, the influence of shear deformation in the panel zone cannot be neglected for external diaphragm joints, which also was concluded in Literatures (Kato et al. 1988; Bao et al. 2011), otherwise, it would lead

Table 7.6 Comparisons of θ and γ

Specimen no.	γ (rad)	θ (rad)	γ/θ (%)
SRJ1-3	0.007192	0.023451	30.67
SRJ2-1	0.007220	0.024246	29.78
SRJ4-2	0.013381	0.034010	39.34
RJ5-1	0.017595	0.043008	40.91

to a large error in the structural analysis. On the other hand, the magnitude of the shear deformation of the specimens at the peak load was lower than 0.02 rad, and the relative beam-to-column rotation was about 0.04 rad, which is also small. Hence, the external diaphragm joint between the CFDST column and the steel beam can still be regarded as the category of strong-joint.

7.5.5 Strain Nephogram in Panel Zone

In contrast with two contiguous images, the displacement in the panel zone surface can be obtained, and then shear strain γ_{xy} at any speckle can be calculated by the software e Vic2D. $\gamma_{xy} = \alpha + \beta$, where α and β are the tangent angular changes in the direction of the x -axis and the y -axis respectively in the micro-element as shown in Fig. 7.27.

In the software, the default positive direction is to right for x -axis and downward for y -axis, so the direction of shear strain in the diagonal regions is opposite, while is the same in fact as shown in Fig. 7.28. As a result of the two failure modes in the test, and shear strain fields in panel zone show two types of different trends too. Figure 7.28a, b present the strain nephogram of these two different specimens such as RJ2-1 under beam failure mode and SRJ5-1 under column failure mode. In order to describe the strain field evolution and development, the strain nephogram are taken at four typical moments including later linear elastic point, yielding point, peak load point and failure point, where failure state means the moment when the lateral load falls to 85% of the maximum lateral strength as same as the description in *ductility*. As can be seen in Fig. 7.28, the strain in the bottom is larger than the top part in every nephogram because of the different internal forces in the upper and lower column ends.

The shear strain nephogram of both specimens are basically in horizontal symmetry in the elastic stage, and gradually develops obliquely, finally in nearly 45° symmetrical distribution. Shear strain is small in latter elastic point, and increases rapidly from the yielding point to peak load point. After the peak load, the shear strain is still increasing since larger lateral displacement is conducted on the column top.

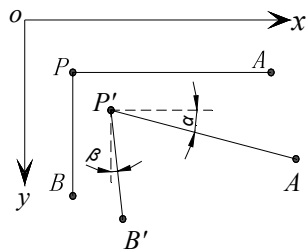


Fig. 7.27 Shear strain calculation at any speckle

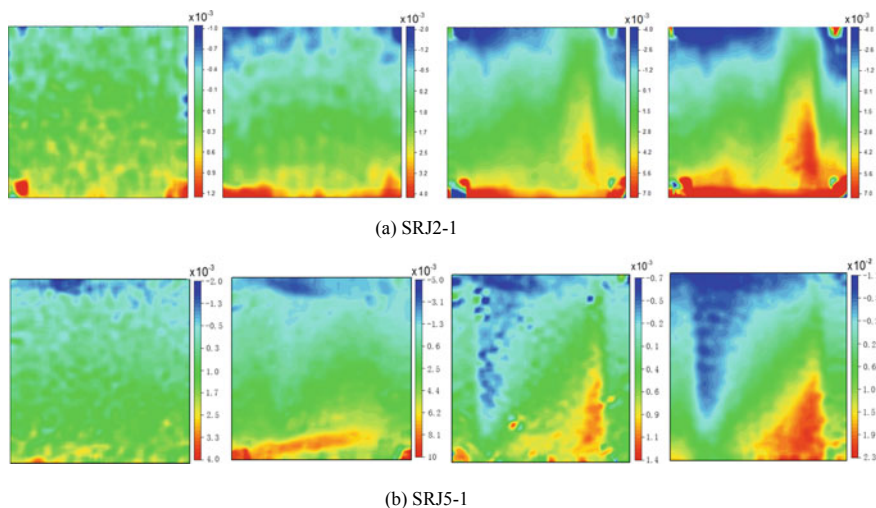


Fig. 7.28 Shear strain field in panel zone

During the whole loading process, the magnitude of shear strain is largest at the four corner points, diminishing towards the center. Shear strain of SRJ5-1 is 2–3 times the magnitude of SRJ2-1 because the panel zone of SRJ5-1 under column failure mode, is subjected to larger shear force. All these data coincide with the strain value obtained by the strain gauge in the experiment. For example, shear strain at central point in panel zone of Specimen SRJ2-1 increased slowly before the peak load, and it kept small under $4000 \mu\epsilon$ all along so panel zone was in elastic stage basically; while shear strain level increased fast after the yielding load, up to nearly $10000 \mu\epsilon$ at the peak load point for Specimen SRJ 5-1. Therefore, panel zone of SRJ2-1 was considered in elastic all along while it entered the elastic-plastic stage for Specimen SRJ5-1. However, the development of shear strain in panel zone was not enough to cause shear failure, which confirmed the design requirements of strong joint & weak member. In general, shear deformation calculated by DSCM with the inclusion of strain nephogram demonstrates the accuracy of the results obtained with artificial vision.

7.6 Summary

Nine external diaphragm joints between CFDST columns and steel beams were tested to investigate the seismic performance and especially two types of failure modes. They were compared with one external diaphragm joint between a square CFST column and a steel beam. A discussion has been presented on the effects of different test parameters (axial load level, width of external diaphragm and bending

stiffness ratio) on the following characteristics of connection performance: ultimate bearing capacity, ductility, stiffness degradation, and energy dissipation. From the test results, the following conclusions are obtained within the limitations of research in this chapter:

(1) External diaphragm joints are improved with consideration of the cross-sectional feature with inner-circular and outer-square steel tubes in the CFDST column. The lateral load (P) versus displacement (Δ) hysteresis loops show a plump shape. It can be concluded that the external diaphragm joint in CFDST structure has high bearing capacity, good ductility and energy dissipation capacity.

(2) The failure mode was mainly dependent on the beam-to-column bending stiffness ratio (k) and the axial compression ratio (n). The ideal failure mode of beam failure can be gotten when k is small, but the limit of n should be considered to guarantee a strong column & weak beam seismic design. Large k results in column failure for the external diaphragm joint in CFDST structure. So, the limit of these two factors should be considered to avoid the column failure in engineering applications design.

(3) The initial stiffness of the external diaphragm joint significantly increased with the improvement of n , but stiffness suffered a sharp degradation; the ductility and energy absorption ability improved with the width improvement of the external diaphragm; ribbed anchorage webs can effectively improve the bearing capacity of the external diaphragm joint. The ribbed joint with large k bore higher maximum lateral load and had larger initial stiffness, whilst had better energy absorption ability.

(4) The digital speckle correlation method (DSCM) was also used to measure and investigate the strain field in panel zone and the relative beam-to-column rotation, which offered better predictions to analyze the mechanical behavior of the external diaphragm joints in CFDST structures. According to the classification of initial rotation stiffness, the external diaphragm joint can be divided into a semi-rigid joint in a sway structure. With the increase of loading, the deformation of joints under column failure mode developed more quickly than the joints under beam failure mode, and the shear strain in the joint domain was 2–3 times the magnitude of the latter one. The shear deformation should not be neglected for it accounts for 30–40° of the relative beam-to-column rotation. But the magnitude of the shear deformation and the beam-to-column rotation at the peak load were both small, so the external diaphragm joint in CFDST structure can still be regarded as the category of strong-joint.

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Chapter 8

Anchored Web Joints with Haunches



Abstract Among the beam-to-CFST column connections, through-beam joint is one effective rigid connection, which has been investigated by experimental study and finite element (FE) analysis (Alostaz and Schneider 1996; Azizinamini and Schneider 2004; Elremaily and Azizinamini 2001; Khanouki et al. 2016; Mirghaderi and Dehghani Renani 2008; Schneider and Alostaz 1998; Sheet et al. 2013). It was concluded that the through-beam joint had reliable connectivity, good plasticity and ductility, strong energy dissipation capacity and high stiffness. However, the continuous beam flange in CFST column is difficult to mount itself in the steel tube and it also affects the pouring of concrete in tube. Therefore, some researchers have proposed and modified a new load transferring mechanism to improve application of continuous plates in steel beam-to-CFST column connections. For example, the through-web joint only using the vertical continuous plate was tested by Chiew et al. (2001), and it was found that the most of shearing force was transferred from steel beam to core concrete through the continuous web, and local buckling of steel tubes due to excessive shearing force was avoided. A vertical continuous plate passing through the column develops a reliable load transfer path by connecting with the beam web (Mirghaderi et al. 2010), and the through-web joint is verified in accordance with the design method specified by AISC. A through stiffener connection was designed between a circular CFST column and an I-beam (Jeddi et al. 2017), and the through stiffener as similar as a vertical through-web was proposed to improve the connection's stiffness and ductility. In the foregoing chapters, vertical stiffener joint and external diaphragm joint with ribbed anchorage plates were introduced, and the ribbed anchor plate was verified to improve seismic behaviors of CFDST structures effectively. Besides, welded haunch joints have also been applied in CFST structure as described in Introductions of this book. The seismic behaviors of steel beam-to-CFDST column connections with haunches were studied by FE modeling (Chu et al. 2009; Dong et al. 2016), and it was found that joints with haunches had higher bearing capacity, stronger energy dissipation capacity and better ductility. Therefore, in consideration of the characteristics of double steel tubes in the CFDST column, the anchored web joint can be designed; meanwhile haunches need to be used to improve stiffness and ductility of the anchored web joint. Four beam-to-column specimens were designed for the low-cycle reciprocating load test to analyze failure modes, bearing capacity, deformation capacity and other seismic behaviors. And FE models

were also established using ABAQUS to verify those test results and to conduct parametric analysis on seismic behaviors of the anchored web joint. The research can provide a scientific basis for design and application of anchored web joints in CFDST structures.

8.1 Test Overview

8.1.1 Specimens Design

The dimensions and details of the anchored web joint specimens (Zhang et al. 2020) are shown in Fig. 8.1. In the process of making specimens, the steel beam was cut off at 100 mm away from the haunch plate. After punching holes in the outer tube,

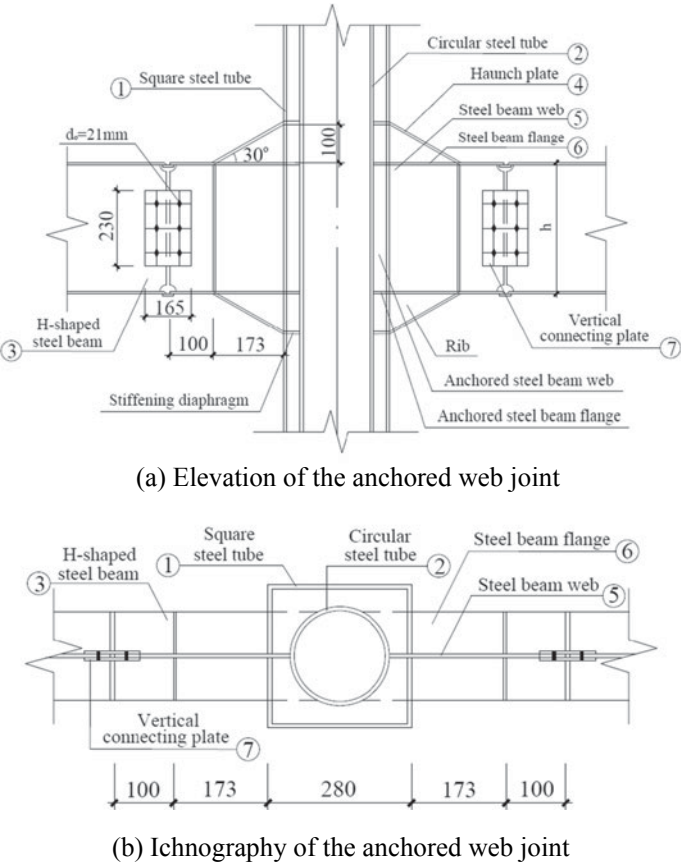


Fig. 8.1 Dimensions and details of specimens (mm)

Table 8.1 Parameters of specimens

Number	Anchored member	Steel beam	k
SPJ1	Steel beam web	H346 $\times$ 174 $\times$ 6 $\times$ 9	0.31
SPJ2	Steel beam web	H350 $\times$ 175 $\times$ 7 $\times$ 11	0.41
SPJ3	Steel beam web and flange	H350 $\times$ 175 $\times$ 7 $\times$ 11	0.41
SPJ4	Steel beam web and stiffening diaphragm	H350 $\times$ 175 $\times$ 7 $\times$ 11	0.41

anchored members were welded on the inner tube, and the anchored members include steel beam web, steel beam flange, and stiffening diaphragm, as shown in Fig. 8.1a. Concrete using coarse aggregate with small particle size was poured into the steel tubes after filling the holes of the outer tube. Finally, the beam flanges were butt welded, and the beam webs were connected by high-strength bolts and connecting plates. The steel beam H346 $\times$ 174 $\times$ 6 $\times$ 9 mm⁴ was used in Specimen SPJ1. The beam-to-column linear bending stiffness ratio (k) was 0.31, and the anchored member was the steel beam web. The steel beam H350 $\times$ 175 $\times$ 7 $\times$ 11 mm⁴ was used in Specimens SPJ2, SPJ3, SPJ4, $k = 0.41$, and the anchored members were taken as different as shown in Table 8.1. Since the thin outer steel tube wall was not beneficial for transferring force, stiffening diaphragms between the outer tube and the inner tube were set to connect the haunch plate using the same steel as the anchored steel beam flange in Fig. 8.1a. The total height of the column was 2070 mm, and the total length of the beam was 3700 mm. The dimension of the inner steel tube was 194 $\times$ 6 mm², and the outer steel tube was 280 $\times$ 10 mm². The axial load on column top was controlled by axial compression ratio of 0.275.

8.1.2 Material Properties

The steel grade used in the test was Q235B, and uniaxial tensile tests of steel plates were conducted according to the specification GB/T 228.1-2010 (2010). The concrete grade was C60, and some 150 $\times$ 150 $\times$ 300 mm³ prism specimens and 150 $\times$ 150 $\times$ 150 mm³ cube specimens were fabricated for axial compression tests according to the specification GB/T 50081-2016 (2016). Material properties of steel and concrete are shown in Table 8.2, in which the measured values of each material are taken as an average of three test samples.

8.1.3 Test Device and Loading System

The test setup and loading system refer to Sect. 7.3.2. A constant vertical load was applied at the column top firstly through a hydraulic jack. Then, a horizontal cyclic load was applied on the side of column top through a horizontal actuator. A loading

Table 8.2 Material properties of steel and concrete

Material	Yield strength (MPa)	Ultimate strength (MPa)	Elastic modulus (MPa)
Square steel tube	337.90	460.38	2.13×10^5
Circular steel tube	343.88	443.45	2.11×10^5
Haunch plate	295.51	406.89	2.13×10^5
Flange of the small beam	288.69	405.41	2.00×10^5
Flange of the large beam	327.64	465.01	2.17×10^5
Concrete	/	60.80	3.61×10^4

cycle was considered as southward pushing-unloading-reversed pulling-unloading. The southward pushing force was defined as a positive direction and northward pulling force was negative. As shown in Fig. 8.2, the vertical hinged supports under north and south beam ends can make beam ends rotate freely and move horizontally. Four supports were arranged to restrict the steel beam from out-of-plane torsion and instability. The horizontal load at column top was controlled by force and displacement successively. The initial loading was controlled by a graded force of 30 kN until it reached 150 kN, and the loading step corresponding to each graded load cycled



Fig. 8.2 Test setup of the anchored web joint

only once. Then, the loading was controlled by displacement with an increment of $0.5 \Delta_y$ per cycling step as similar as the loading control of the external diaphragm joint in the last chapter.

8.1.4 Arrangement of Measuring Points

The load-displacement hysteresis loop was automatically collected by an electro-hydraulic servo loading system, and displacement and strain were automatically collected by an IMP data system. The same arrangement schedule was adopted for each specimen. The in-plane displacements were measured by eight displacement transducers as shown in Fig. 8.3a. Transducers D1~D4 were located on the outer steel tube surface to measure absolute lateral displacement for the lower and upper column. h is the overall beam height, then D1, D4 were located at the point of $1 h$ from the beam flange surface, and D2, D3 at the point of $0.5 h$. D5, D6 were used to measure shear deformation in panel zone; D7, D8 were used to measure beam-to-column relative rotation. Unidirectional strain gauges were arranged on the steel surface, as shown in Fig. 8.3b, where circumferential and vertical unidirectional strain gauges were arranged at the upper and lower positions of inner and outer tubes respectively. There were strain rosettes at the corner and center points in panel zone.

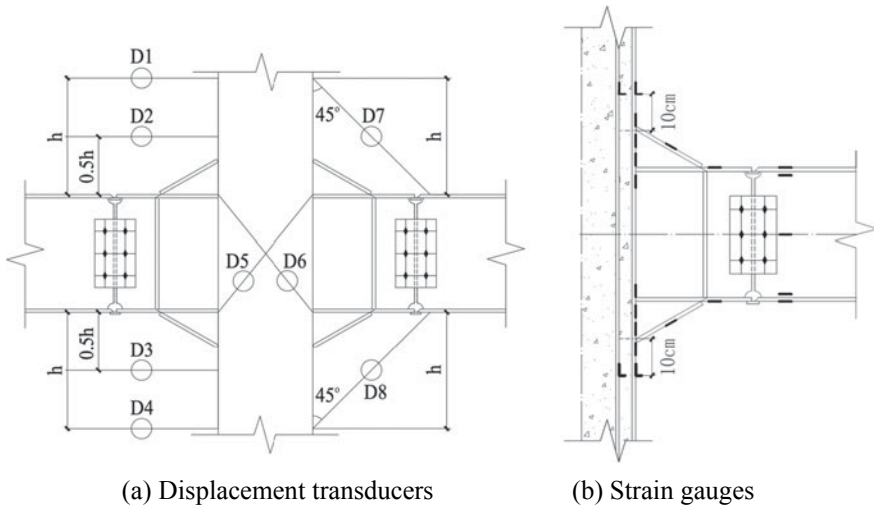


Fig. 8.3 Arrangement scheme of displacement meters and strain gauges

8.2 Numerical Analysis Model

The FE models with the same loading condition as the test were established by software ABAQUS. Reasonable material constitutive relationships were selected, and friction was considered in contact plane between the hydraulic jack and the column top.

8.2.1 Constitutive Model

Isotropic and kinematic elastoplastic strengthening model was adopted for the constitutive relationship of steel. This model considers the Bauschinger effect, and the Poisson's ratio is 0.27 according to the material tests. The stress-strain relation of steel and concrete refers to Sect. 6.6.1. Elastic modulus of concrete is taken from the material property test, and the Poisson's ratio is 0.2.

8.2.2 Element Selection and Division

The element type was selected as C3D8R. The element was divided by the structural adaptive method, and the influence of mesh size on the contact surface was considered. The hexahedron advanced algorithm was used and the meshes around circular holes were densified due to stress concentration.

8.2.3 Interactions, Boundary Conditions and Loading Modes

"Tie" constraint was applied for simulating welding among components in FE model, and influences of initial defects, welding defects and welding residual stress on performances of joints were ignored. The contact between the steel tube and concrete was surface-to-surface contact. The penalty function was adopted in the friction formula of the tangential behavior, and the "hard contact" was adopted in the normal behavior. The friction coefficient was taken as 0.6 according to the analysis method proposed by Han et al. (2015). Due to the symmetrical boundary and loading conditions, only a 1/2 model was established, as shown in Fig. 8.4. According to actual loading procedure in the test, two analysis steps were set to simulate axial compression and horizontal cyclic loading at column top.

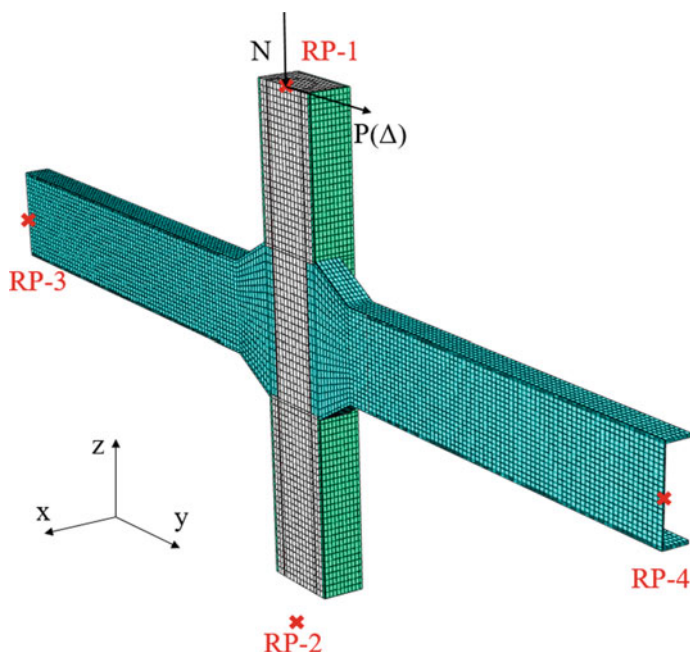


Fig. 8.4 Element division and loading mode of the half-model

8.3 Comparison Between Test and Numerical Simulation Results

8.3.1 Destruction Process

After the column slightly settled under axial compression applied at the column top, lateral supports and vertical hinged supports at the north and south beam ends were fixed, and a low-cycle reciprocating load was applied at the column top. FE analysis could help us to better comprehend stress distribution or deformation development in the whole loading process. The test phenomena during the loading processes are shown in Fig. 8.5, together with stress nephograms from FE simulation. When the force on the column top reached 150kN, the measured strain values were basically below $1000 \mu\epsilon$, which indicated that the specimens were still in elastic state. During the displacement-controlling process, when the displacement of the column top reached 15~20 mm in the test, the strain of steel tube close to the lower haunch plate increased to the yield strain of $1500 \mu\epsilon$. When the displacement of SPJ2 reached 20 mm in FE simulation, the stress of this part as shown in Fig. 8.5a also exceeded the yield strength. When the displacement reached 24~28 mm in the test, the upper flange surface of the steel beam peeled as shown in Fig. 8.5b. Similarly, in FE model SPJ2, the stress value of the beam flange near the haunch plate was large when the

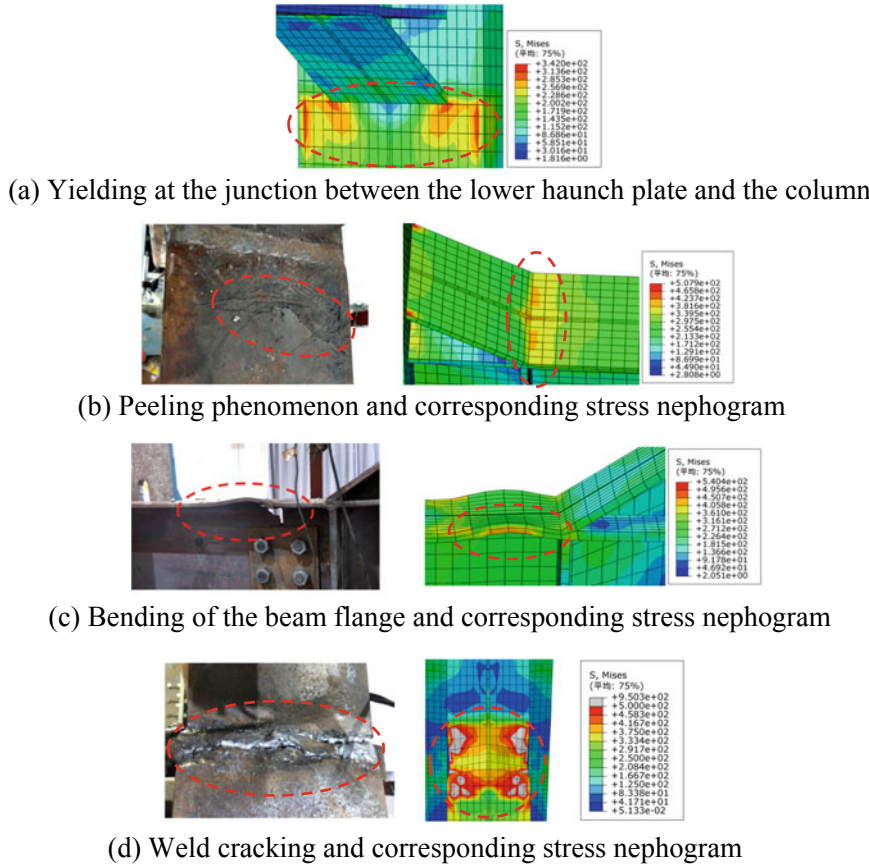


Fig. 8.5 Loading processes in test and FE analyses

displacement was over 30 mm. When the displacement increased to 38~42 mm in the test, the upper flanges of steel beams gradually bulged, as shown in Fig. 8.5c. The bending of the steel beam flange occurred when the displacement reached 40 mm in the FE simulation for Specimen SPJ2. When the displacement reached 43~55 mm in the test, the local beam flange bent continuously to form a plastic hinge. Finally, when the displacement reached 52~65 mm in the test, some cracks appeared at the butt weld of the steel beam flange as shown in Fig. 8.5d, so the butt weld was prone to generate stress concentration or be brittle due to construction quality problems, and haunches were effective to avoid a damage at the beam-to-column junction. FE models didn't simulate the weld crack, but it can be found in the stress nephogram that the stress at this zone far exceeded ultimate strength of the material. FE modeling was halted when the horizontal load at the column top had been reduced by about 20% of the peak load, and the ultimate horizontal displacement herein was basically larger than 70 mm. It can be concluded that the main failure mode of the anchored

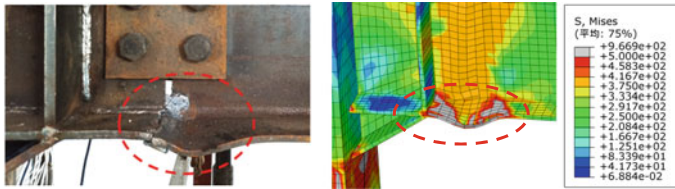


Fig. 8.6 Comparison of the failure modes between test and FE analyses

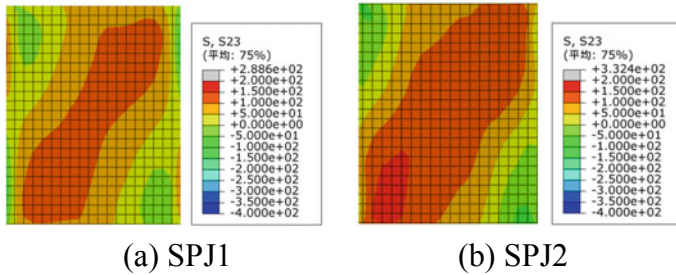


Fig. 8.7 The shear stress nephograms in panel zone

web joint was a plastic hinge formed at the beam end as shown in Fig. 8.6. By attaching haunches in the beam-to-column connection, plastic hinges at the beam end were moved to the cross-section corresponding to the tip of the haunch.

At the point of peak load, shear stress distributions in the panel zone of Specimen SPJ1 with a small steel beam and SPJ2 with a large steel beam are shown in Fig. 8.7. The shear stress is obliquely symmetrical in 45° , and becomes larger for SPJ2 owing to an increase of beam-to-column linear bending stiffness ratio. But in general, since failure modes of the anchored web joint obtained by both tests and numerical simulations were all plastic hinges appeared at the beam ends, and the value did not reach yield strength in the shear stress nephogram. The result showed that the anchored web joint with haunches conforms to the engineering requirements of strong joints.

8.3.2 Strain and Stress Responses

There were circumferential and vertical unidirectional strain gauges pasted on the upper and lower positions of outer and inner steel tubes respectively. The strain of the outer tube presented no much difference for all the four joint specimens, but the strain of inner tube is very different, such as Specimens SPJ2 and SPJ4. The horizontal load (P)-strain (ϵ) skeleton curves are shown in Fig. 8.8. Besides the same anchor plate as in Specimen SPJ2, there were additional anchored stiffening

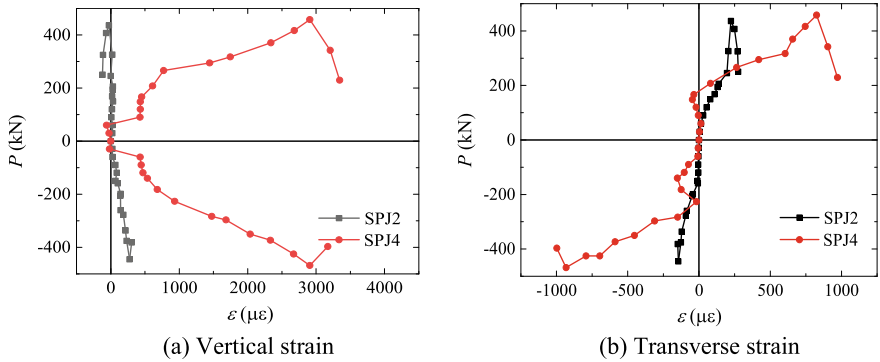


Fig. 8.8 Strain of inner steel tube in the test

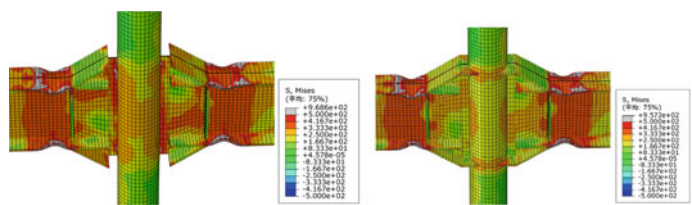


Fig. 8.9 Stress nephograms of joints core zone

diaphragms in Specimen SPJ4. For SPJ2, the inner tube is totally in elastic state, while the strain value of SPJ4 is much larger than that of SPJ2. Because stiffening diaphragms can help to transfer internal force into joint core, the connected inner tube takes function in SPJ4. Moreover, vertical strain is much higher than transverse strain since circumferential restrict is larger than the longitude direction. According to stress distribution at the failure state from the FE simulation as shown in Fig. 8.9, anchored stiffening diaphragms between the double steel tubes in SPJ4 make stress distribute more evenly than in SPJ2, and stress concentration is obvious for SPJ2. Therefore, the anchored web joint can give full play to sectional characteristics of the double steel tubes and exhibit excellent structural internal force redistribution. As can be seen, the plastic hinge forms at the beam end under the condition of principle “strong column & weak beam”, so the anchor joint with only steel beam web is enough to realize a strong connection.

8.3.3 Hysteresis Loops

The comparisons of horizontal load (P)-displacement (Δ) hysteresis loops obtained by the test and FE analysis are shown in Fig. 8.10. The numerical simulation results are in good agreement with the test results. The hysteresis loops of the anchored web

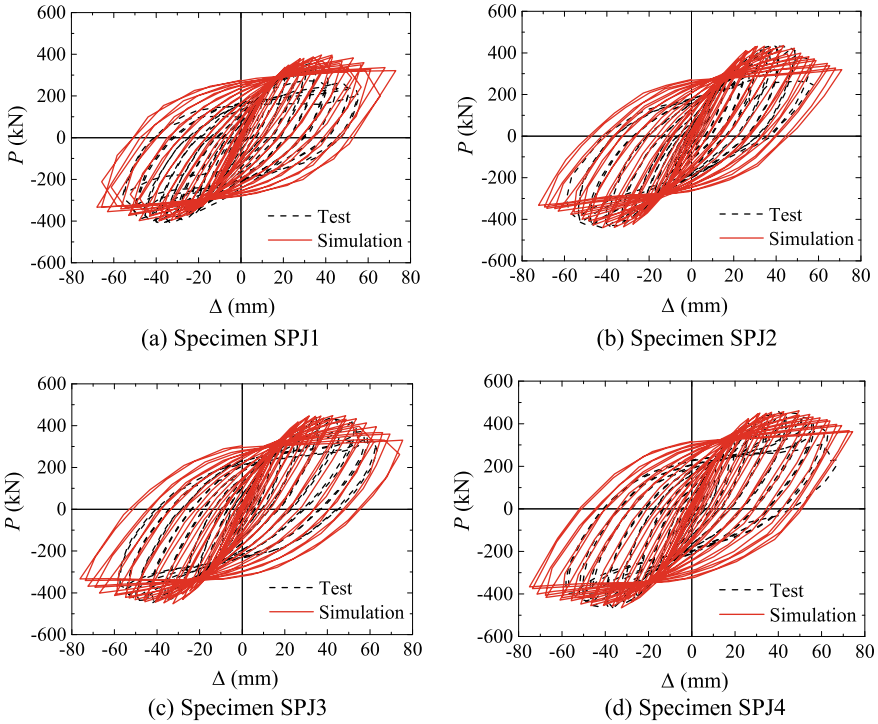


Fig. 8.10 Comparisons of load (P)-displacement (Δ) hysteresis loops between test and FE analyses

joint can be divided into three stages. (1) Elastic stage. The joint stiffness is basically unchanged, and the P - Δ relationship maintains in linear elasticity. (2) Deformation development stage. The hysteresis loop is gradually full and the pinching phenomenon is not obvious, indicating that anchored web joints have good energy dissipation capacity. The stiffness gradually degenerates with the increase of loading displacement, and in the FE analysis we found a gradual increase in the bending curvature of steel beams and the deformational areas of CFDST columns. The horizontal displacement corresponding to the peak load was around 40 mm for anchored web joints. This indicates that the connection stiffness of the anchored web joint is very strong, so the allowable deformation is small, which results in the stress concentration on weak part of welds. (3) Failure stage. After the peak load, the experimental load decreases rapidly with the increase of horizontal displacement, while the numerical simulation curves keep decreasing slowly. The curves obtained from the numerical simulation are symmetrical on the pushing and pulling directions.

Bearing capacity and ductility of the test and numerical simulation results are shown in Table 8.3. The average error of the ultimate bearing capacity between the test and FE simulation is 2.24%, and the maximum error is 5.2%. Ductility coefficient (u) is defined as a ratio between failure displacement and yield displacement, and the yield displacement is determined by the yield moment method (Wang et al. 2007).

Table 8.3 Bearing capacity and ductility of specimens

Specimen			Yield point		Peak point		Failure point		u
			P_y (kN)	Δ_y (mm)	P_m (kN)	Δ_m (mm)	P_u (kN)	Δ_u (mm)	
SPJ1	Simulation		349.4	23.3	397.3	41.4	321.0	73.0	3.13
	Test	Push	282.4	19.4	345.6	34.5	293.7	40.9	2.11
		Pull	369.8	25.1	410.0	37.6	348.5	51.8	2.06
SPJ2	Simulation		405.3	25.3	436.3	43.6	320.0	71.0	2.81
	Test	Push	370.9	26.3	436.8	41.0	371.3	55.0	2.09
		Pull	393.0	27.6	406.2	44.1	345.0	58.3	2.11
SPJ3	Simulation		415.3	26.3	447.7	48.6	330.3	75.3	2.86
	Test	Push	385.9	25.0	443.4	36.3	376.2	55.7	2.23
		Pull	382.6	26.2	450.0	43.6	382.5	56.2	2.15
SPJ4	Simulation		417.8	26.6	460.5	49.6	362.6	74.4	2.80
	Test	Push	395.3	26.1	457.1	40.6	388.5	59.1	2.26
		Pull	418.9	25.9	463.5	39.8	394.0	57.1	2.20

The average ductility coefficient obtained by test is 2.15, which is small because welding fractures occurred earlier after bending of the steel beam. It is very necessary to ensure weld quality and minimize heat zone in practical engineering. So, in the hysteresis loops obtained from FE analysis, there is a longer descending phase after the peak load, and the average ductility coefficient obtained by FE analysis is 2.9, then a better ductility behavior of the anchored web joint can be verified. The bearing capacity of Specimens SPJ3 and SPJ4 with more anchored members is only about 5% higher than that of Specimen SPJ2, and the improvement is not obvious because the anchored web joint with haunches can guarantee transmission of shear force and bending moment from the beam end to joint core zone effectively. Unless the strength of steel beam is very strong, the more anchored members are needed to ensure the rigid connection, which will be discussed in the following parametric analysis.

8.3.4 Joint Energy Dissipation Capacity

Although improvement of bearing capacity of Specimen SPJ3 and Specimen SPJ4 with more anchored members was not obvious, the energy dissipation capacity of the joints has been obviously improved. The energy dissipation capacity of joint specimens is evaluated by equivalent viscous damping coefficient h_e (Tang 1989) and half-cycle energy dissipation E_t (take the positive hysteresis loop). The relationship between the equivalent viscous damping coefficient and the half-cycle number of each specimen is shown in Fig. 8.11. In this chapter, the equivalent viscous damping coefficients $h_e = 0.273 \sim 0.334$ when anchored web joints reached the peak point, and the equivalent viscous damping coefficient reached a higher value when each

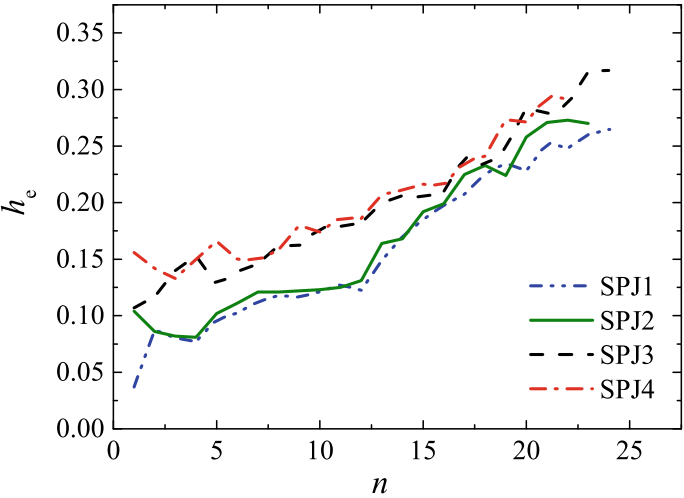


Fig. 8.11 Relationship between equivalent viscous damping coefficient (h_e) and half-cycle number (n)

specimen failed. The relation between energy dissipation in a half-cycle (E_t) and half-cycle number (n) of each specimen is shown in Fig. 8.12. With the increase of the cycle number, the energy dissipation capacity increases gradually. Especially, the energy dissipation performance of SPJ3 and SPJ4 is better than that of SPJ2, and the additional anchored members are beneficial to enhance the energy dissipation

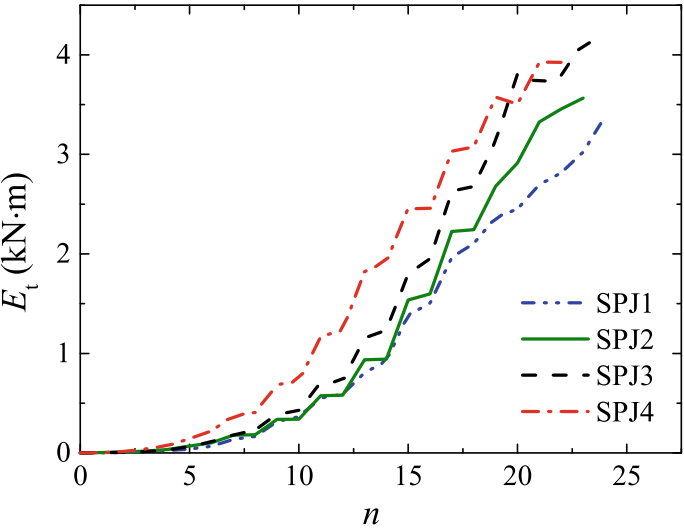


Fig. 8.12 Relationship between energy dissipation in a half-cycle (E_t) and half-cycle number (n)

performance of the joint. In terms of the stress distribution in stress nephogram, as shown in Fig. 8.9, the deformation region at the ultimate bearing load for SPJ4 is wider than that of SPJ2, so the energy dissipation capacity improved.

8.4 Parametric Analysis

It can be concluded from foregoing comparisons that the numerical simulation results were in good agreement with the test results. So more numerical models were built to analyze main parameter influences on seismic behaviors of anchored web joints in CFDST structures, such as failure mode, bearing capacity and deformability. The selected parameters contain structure construction (anchored members, ribs, haunches), geometric parameters (beam-to-column linear bending stiffness ratio) and material parameters (steel beam strength, concrete strength). In addition, only the axial compression ratio of 0.275 was adopted due to the limit of test conditions, so the influence of axial compression ratio was also studied by the numerical simulation. Considering convenience of construction and pouring concrete in the column, the anchored web joint without other anchored members was applied in the FE simulation. Based on the test specimen SPJ2, the parameters for numerical simulation are shown in Table 8.4. Since the curves obtained by the FE analysis is symmetrical, the first quadrant of P - Δ skeleton curve is selected for numerical analysis.

8.4.1 Influence of Joint Construction

In the test, four specimens were equipped with ribs and haunches to ensure the joint core zone intact. In the FE simulation, mechanical performance of the joints without

Table 8.4 Parameter settings

Types	Parameters	Details
Structural parameters	Both ribs and haunches	SPJ2 ^a
	Only ribs	SPJ2-H
	No ribs and haunches	SPJ2-HS
Geometric parameters	Beam-to-column linear bending stiffness ratio (k)	0.31, 0.41 ^a , 0.47, 0.7
Material parameters	Steel tube strength	Q235 ^a , Q345, Q420, Q550
	Steel beam strength	Q235 ^a , Q345, Q420, Q550
	Concrete strength	C40, C50, C60 ^a , C80, 0
Compression on column top	Axial compression ratio (n)	0.04, 0.275 ^a , 0.6

Annotation ^arepresents parameters of the test Specimen SPJ2

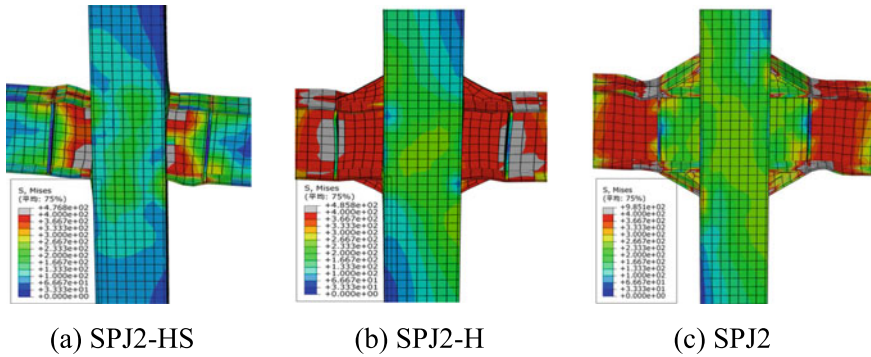


Fig. 8.13 Failure modes of joints with different joint constructions

ribs or haunches was analyzed. At the base of test Specimen SPJ2, haunches were not established in the model SPJ2-H, so there were ribbed anchor plates as the force transferring member in the beam-to-column connection; while the model SPJ2-HS was a pure anchored web joint, there were no reinforcing members. Therefore, failure modes of SPJ2-HS, SPJ2-H and SPJ2 are very different, as shown in Fig. 8.13. In the final state of SPJ2-HS, local buckling occurred at the column tube near the beam-to-column junction owing to stress concentrations. Because the bending resistance of steel beam was much lower than the CFDST column, the steel beam flange bent to form a plastic hinge very close to the junction. Especially the connection stiffness of SPJ2-HS was small, and the deformation focused on beams and columns certainly. The stress concentration at the junction of SPJ2-H is alleviated after installing ribs, so the ribs connect with the steel beam effectively, then the steel buckling is reduced for the CFDST column and buckling position is moved outside for the steel beam. But the stress is very large at the connection between the rib plate and steel beam in SPJ2-H, then as for SPJ2, this phenomenon is avoided owing to both ribs and haunches attached to the anchored web joint. Therefore, an ideal failure mode that the plastic hinge appeared at the beam end is respected totally, and the engineering requirement of “strong column & strong joint” is achieved for anchored web joint with haunches. Figure 8.14 shows P - Δ skeleton curves of the joints with different structures. The initial stiffness of SPJ2 is 17.0% higher than that of SPJ2-H, and 36.9% higher than SPJ2-HS. The ultimate bearing capacity of SPJ2 is increased by 24.3 and 42.1% compared with that of SPJ2-H and SPJ2-HS respectively.

8.4.2 Influence of Steel Beam Strength

The bending strength ratio of column to beam in the test was 2.08, so we select stronger steel as Q345, Q420 and Q550 for the beam in the simulation, and the ratio is changed as 1.84, 1.75 and 1.31 respectively. The P - Δ skeleton curves of the anchored

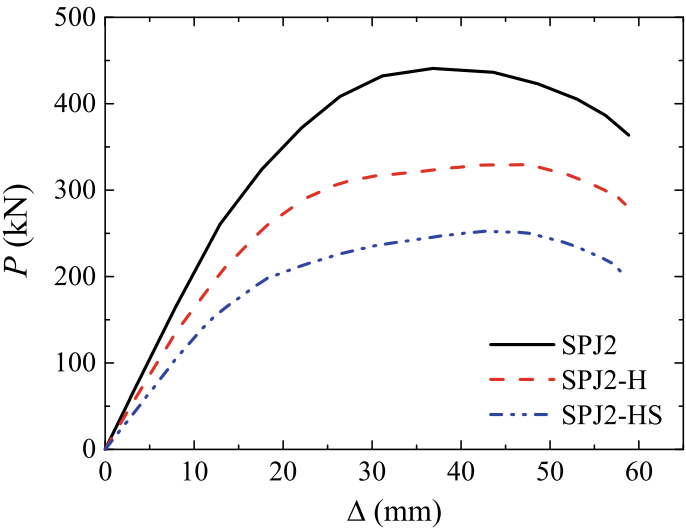


Fig. 8.14 P - Δ skeleton curves of joints with different joint constructions

web joints with different steel beam strength including the test specimen with Q235 steel beam are shown in Fig. 8.15. As can be seen, yield points move backward and the ultimate bearing capacity is improved with the increase of steel beam strength. But initial stiffness of each joint was basically the same due to same cross-section and elastic modulus of the steel beam. The ultimate bearing capacity of the joints

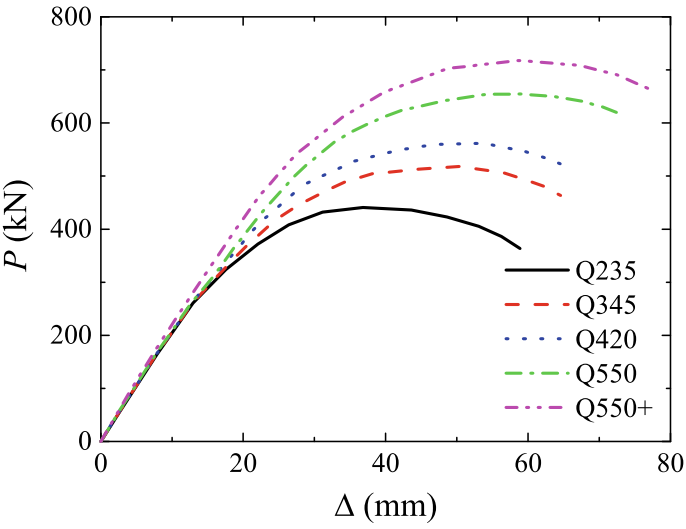


Fig. 8.15 P - Δ skeleton curves of joints with different steel beam strength

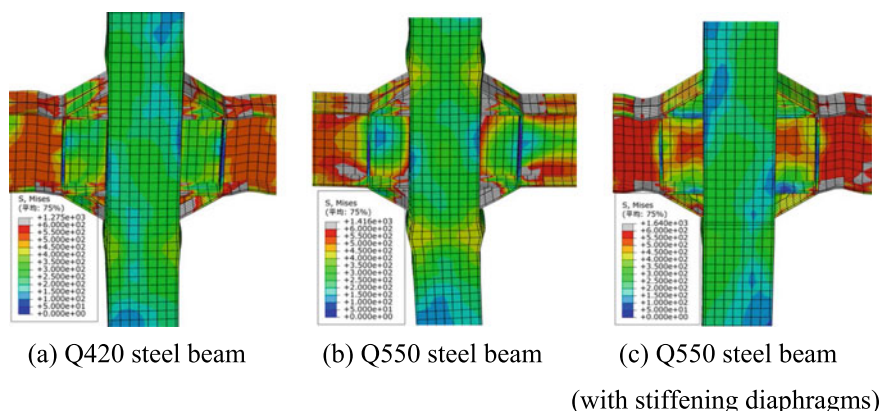


Fig. 8.16 Failure modes of joints with different steel beam strength

with Q345, Q420, Q550 steel beam was respectively 18.8, 28.9, 49.8% higher than that of the joint with Q235 steel beam. According to the stress nephograms in FE simulation, stress in outer steel tubes increased obviously with the increase of steel beam strength, which resulted in increasing deformation in outer steel tubes. The failure modes of the joints with Q420 and Q550 steel beam are shown in Fig. 8.16a, b. As for the joints with Q345 and Q420 steel beam, the plastic hinge still occurs at the beam end as same as the joint with Q235 in the test, but there is following slight bulge at the column wall due to stress concentration. However, the failure mode is changed to the plastic hinge at the column end for the joint with Q550 steel beam, as shown in Fig. 8.16b. Although the column-to-beam bending strength ratio is larger than 1, the tensile force from the beam flange is first transferred to the outer steel tube, and then to the inner steel tube and the core concrete through the anchored beam web. The function of the inner steel tube and concrete is lagged, and similarly, the outer steel tube is adopted as Q235 steel, so the bulge of column wall occurs firstly near the haunch plate, then the inner steel tube and concrete fail successively resulting in a plastic hinge on column. Therefore, in practical engineering, it is better to select a reasonable steel strength ratio of column to beam.

On the other hand, when the bending strength ratio of column to beam is smaller, it will be feasible to apply stiffening diaphragms as the additional anchored member between two steel tubes. According to connection construction of test Specimen SPJ4, a joint model named Q550 + is established with stiffening diaphragms on the base of the joint with a Q550 steel beam, and its final failure mode is shown in Fig. 8.16c. It can be seen that the deformation concentrates on the beam again without bulges on outer steel tube. This attributes to overall working of the whole CFDST column since stiffening diaphragms connect the inner steel tube and concrete with the outer tube solidly. The internal force, especially the tensile force can be sufficiently transferred to the whole column, and there is little stress concentration in the outer steel tube. Furthermore, the plastic hinge is transformed on the steel beam because the beam has a smaller bending strength than the column after all. The bearing capacity

of the joint with stiffening diaphragms was 9.5% higher than that of the joint without stiffening diaphragms since stiffening diaphragms take full advantage of the cross-section of double tubes in CFDST column. Therefore, it will be feasible to apply stiffening diaphragms as the additional anchored member between two steel tubes when the bending strength ratio of column to beam is smaller than 1.3. Otherwise, the anchored web joint is a viable connection owing to the “strong column & weak beam” design requirement.

8.4.3 Influence of Beam-to-Column Linear Bending Stiffness Ratio (K)

In the test, Specimens SPJ1 and SPJ2 had different beam-to-column linear bending stiffness ratio due to different cross-sections of steel beams. The horizontal force was applied on the side of the column top, so the bending moment (M) at the beam end can be calculated using the equilibrium equation. The beam-to-column rotation at every moment was obtained by measurements in the test. Then, the comparison of bending moment (M)-rotation (θ) curves between SPJ1 and SPJ2 is shown in Fig. 8.17. The maximum bearing capacity of SPJ2 with a larger k is increased obviously than Specimen SPJ1 with a smaller one, but the energy dissipation capacity and the ductility are basically the same.

The beam-to-column linear bending stiffness ratio (k) was small in the test, and the change of k can improve stress condition of the anchored web joint. k in the numerical models was taken as 0.31, 0.41, 0.47 and 0.7 respectively. It was controlled by the dimension of beam cross-section for $k = 0.31, 0.41, 0.47$, so for these three models, the total length of each beam was 3700 mm, and cross-section dimensions of the steel beams were $H346 \times 174 \times 6 \times 9 \text{ mm}^4$, $H350 \times 175 \times 7 \times 11 \text{ mm}^4$, and $H396 \times 199 \times 7 \times 11 \text{ mm}^4$ respectively. When $k = 0.7$, it was controlled by the beam length, so for this model, the total length of the beam was 2480 mm, and cross-section

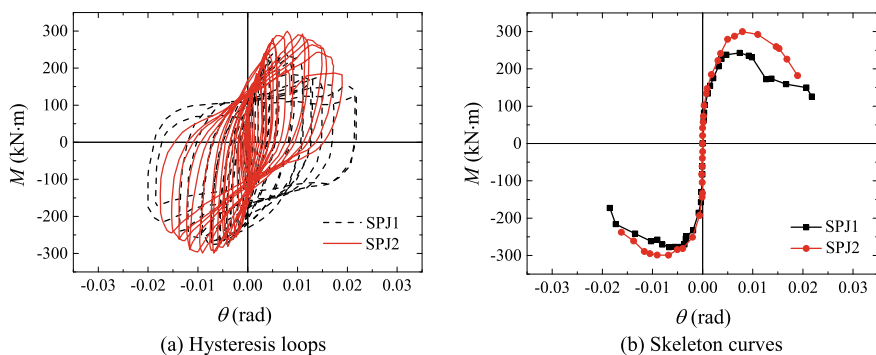


Fig. 8.17 Comparisons of M - θ hysteresis loops and skeleton curves

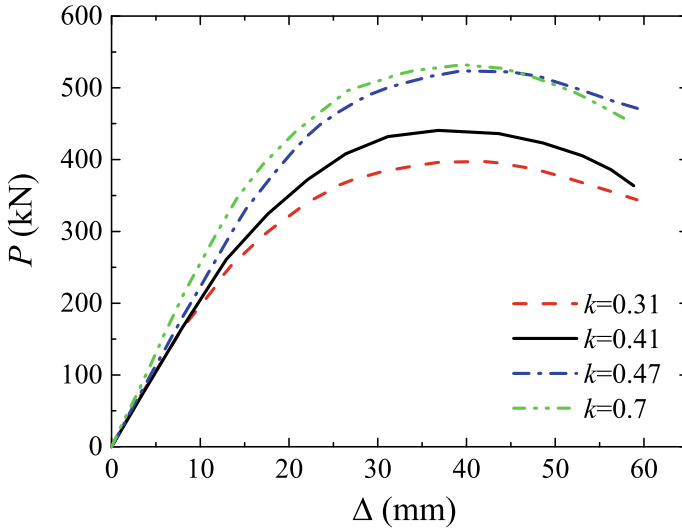


Fig. 8.18 P - Δ skeleton curves of joints with different k

dimension of the steel beam was $H396 \times 199 \times 7 \times 11 \text{ mm}^4$. The FE analysis showed that plastic hinges formed at the beam end for all these four models. P - Δ skeleton curves of the anchored web joints with different k are shown in Fig. 8.18. It can be seen from the figure that the ultimate bearing capacity of the anchored web joints with $k = 0.41$ and $k = 0.47$ is 9.8 and 31.8% higher than that of the joint with $k = 0.31$. Initial stiffness of the anchored web joint with $k = 0.7$ is 16.0% higher than that of the anchored web joint with $k = 0.47$, and change of the ultimate bearing capacity is not obvious. k controlled by cross-section of the steel beam has a great influence on stiffness in the elastic-plastic stage and ultimate bearing capacity, because bending strength and stiffness are improved with the increase of cross-section dimension of the steel beam. k controlled by beam length has a great influence on initial stiffness, but has no influence on ultimate bearing capacity. This attributes to that shortening of beam length decreases radius of rotation with no change of the beam strength.

8.4.4 Influence of Concrete Strength

In FE models, C40, C50, C60, C80 and no concrete are selected for different models. The obtained P - Δ skeleton curves of the joints are shown in Fig. 8.19. The axial compressive force is adjusted to the same axial compression ratio 0.275 as in the test. As can be seen, the joint without concrete in the column had much lower stiffness and bearing capacity, because concrete poured into double steel tubes in CFDST column improved stiffness of the column and local stability of the beam-to-column connection. However, for the CFDST column, the increase of concrete strength had

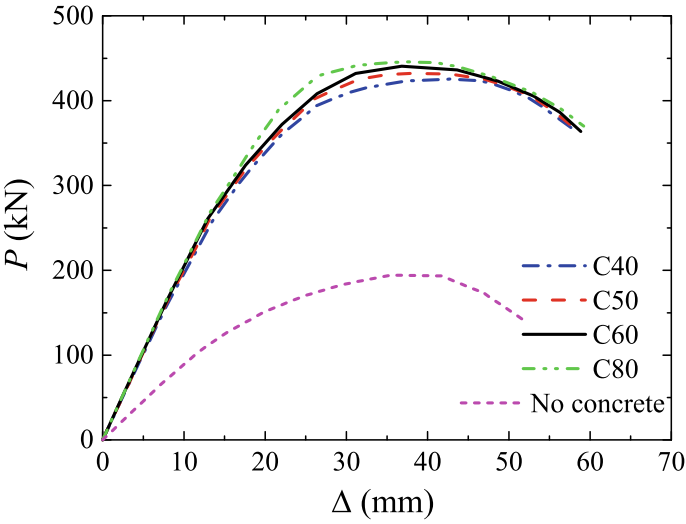


Fig. 8.19 P - Δ skeleton curves of joints with different concrete strength

a little influence on initial stiffness and ultimate bearing capacity, because under the condition of “strong column & weak beam”, plastic hinges formed at the beam end for all the models. The initial stiffness and the ultimate bearing capacity of the joint with C40 concrete were respectively improved by 133.1% and 126.8% than that of the joint without concrete. The failure modes of the anchored web joints with C40 concrete and without concrete are shown in Fig. 8.20. The analysis result of the joint without concrete shows that buckling appears at the junction between the haunch plate and the outer steel tube especially when compressive force acts on the

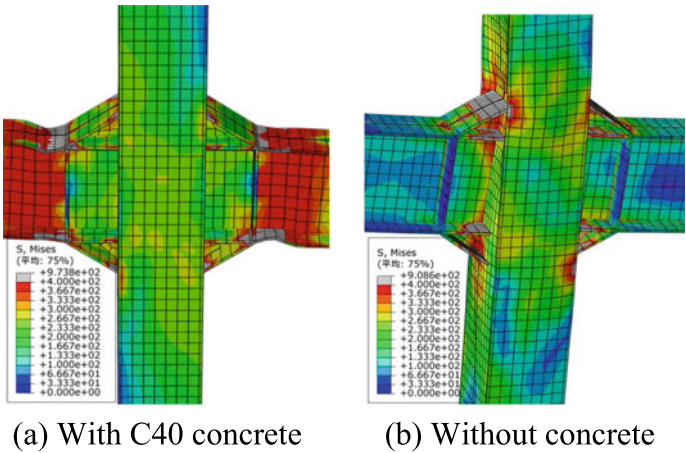


Fig. 8.20 Failure modes of joints with different concrete strength

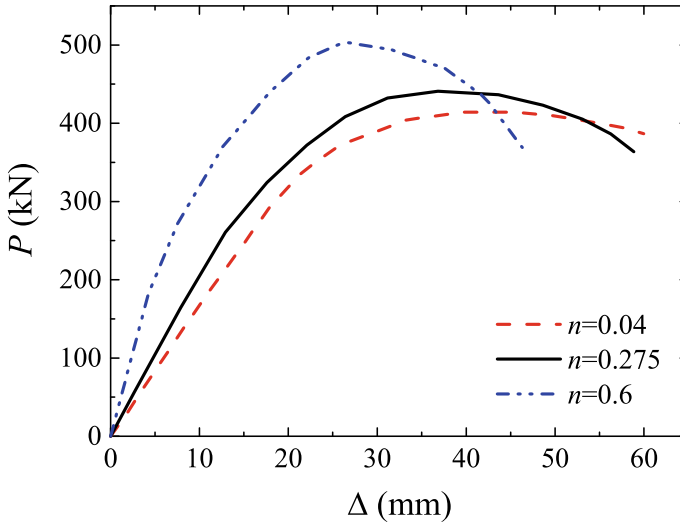


Fig. 8.21 P - Δ skeleton curves of joints with different n

outer steel tube, because thin-walled steel tube cannot bear higher compression. This indicates that anchored web joints only fit for the CFDST columns and it is feasible to reduce concrete strength grade to save cost according to engineering needs.

8.4.5 Influence of Axial Compression Ratio

In the test, the axial compression ratio (n) was 0.275, but mechanical performance of the joint would be affected by axial pressure loading on joint core zone. Then, n was taken as 0.04, 0.275, 0.6 for each FE model. P - Δ skeleton curves of the anchored web joints with different n are shown in Fig. 8.21. The axial compression ratio had a great influence on initial stiffness, ultimate bearing capacity and deformation capacity of anchored web joints. The initial stiffness of the joints with $n = 0.275$ and 0.6 increased by 2.3 and 16.9% than that of the joint with $n = 0.04$. The ultimate bearing capacity increased by 9.0 and 20.0%. Meanwhile, the downtrend in descending section of the P - Δ skeleton curves is more obvious with the increase of n , so the deformation capacity is reduced. Since friction function was considered in the contact plane between the hydraulic jack and the column top in the FE simulation, the higher friction due to the larger axial compressive force increases the initial stiffness and the ultimate bearing capacity. Also, the horizontal load goes down dramatically after peak value owing to the larger axial compressive force.

The failure mode of anchored web joints with $n = 0.04$ and 0.275 is plastic hinges appeared at beams ends, but the failure mode of the joint with $n = 0.6$ is bending failure at column end, as shown in Fig. 8.22. With the increase of axial

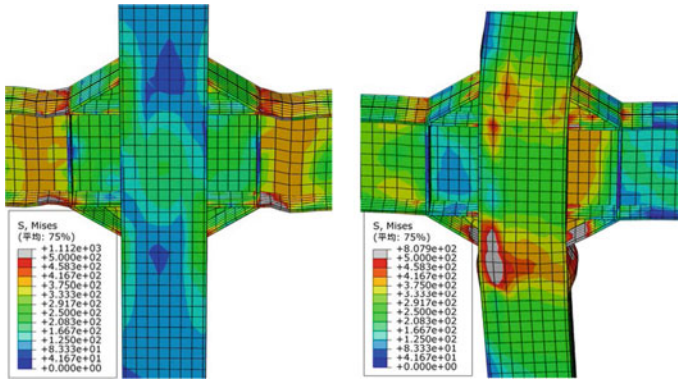


Fig. 8.22 Failure modes of joints with different n

compression ratio, the outer column wall near the haunch plate was easy to be bulged by compression and bending moment from the horizontal load at the column top. Therefore, it is better to control the limit of axial compression ratio to avoid bending failure of the CFDST column in engineering applications.

To sum up, material's yielding position in the anchored web joint can be changed after removing haunches and ribs, and initial stiffness and ultimate bearing capacity of the anchored web joint are reduced dramatically. The ultimate bearing capacity of the joint can be significantly improved by increasing steel beam strength with the same cross-section dimension. The yield strength and the ultimate bearing capacity of the anchored web joint with Q345 steel beam were 46.8 and 18.8% higher than the anchored web joint with Q235 steel beam. The increase of beam-to-column linear bending stiffness ratio controlled by cross-section dimension of steel beam can significantly improve the ultimate bearing capacity of the joint. The increase of beam-to-column linear bending stiffness ratio controlled by beam length can improve the joint initial stiffness, but bearing capacity was not evidently improved. The increase of concrete strength has no obvious effect on initial stiffness and bearing capacity of the joint. The increase of axial compression ratio can improve initial stiffness, bearing capacity but reduce ductility of the anchored web joint.

8.5 Summary

In this chapter, four anchored web joints consisting of the CFDST column and the H-shaped steel beam were tested under the low-cycle reciprocating load. Considering the material constitutive relation, boundary conditions, friction coefficient and complex contact problems, the anchored web joints were analyzed by ABAQUS, and the main conclusions were obtained as follows:

- (1) The main failure mode of the anchored web joint is plastic hinges at the beam ends. The connection stiffness of the joint is strong and deformation of joints concentrates on steel beams. The hysteresis loop is full and the pinching phenomenon is not obvious, indicating that anchored web joints have good energy dissipation capacity. The setting of stiffening diaphragms and anchored steel beam flanges is beneficial to improve energy dissipation performance of the anchored web joint. For the joint with a small ratio of column-to-beam strength, the stiffening diaphragms can improve the overall working performance of the whole CFDST column.
- (2) The FE analysis results were in good agreement with the test results. Parametric analysis on the seismic behaviors was also carried out using the FE models in consideration with connection construction, material and geometric parameters, axial compression ratio. And the analysis results show that axial compression ratio and beam-to-column linear bending stiffness ratio controlled by beam length have a great influence on initial stiffness of the joint. The steel beam strength, the beam-to-column linear bending stiffness ratio controlled by beam cross-section dimension and axial compression ratio have a great influence on bearing capacity. The deformation capacity of the anchored web joints is more affected by axial compression ratio. The increase of steel beam strength and axial compression ratio can lead to bending failure at the column end.
- (3) Through the test and the FE simulation analysis, it was proved that the anchored web joint had good bearing capacity, initial stiffness and ductility. It is better to adopt anchored web joint with haunches, which has good seismic behaviors and it can be applied to the composite frame structure in high-intensity earthquake regions.

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Chapter 9

Blind Bolted T-Plate Joints in Prefabricated Construction



Abstract Comparing to modern reinforced concrete columns, the CFDST column can be constructed without additional supporting equipment during casting concrete, so it is also a good choice for prefabricated buildings to realize the rapid construction. Then, the research on the connection between the steel beam and the CFDST column in a prefabricated structure is of great significance. Bolted joint is a common typical connection type in CFST structures, as shown in Sect. 1.4.7. Those series of bolted end-plate connections of CFST columns indicate that the required performances including strength, stiffness, ductility and failure mode can be achieved by controlling the tube wall thickness and concrete strength, so the connection can appropriately offer good seismic behaviors for its potential use in high-rise buildings. Since the through-bolted joints limit rotational deformation capacity of the end-plate or T-plates, some scholars used blind bolted joints. Zhang (2012) carried out the seismic performance test and finite element simulation of the end-plate blind bolted joint between a CFST column and an H-shaped steel beam. It was found that end-plate thickness and column section type had a great influence on the bearing capacity, stiffness, energy dissipation capacity and other seismic behaviors. Experiments and finite element analysis of end-plate blind bolted joints between CFST columns and steel beams were carried out by Li et al. (2015), Wang et al. (2010), Yao et al. (2008), Wang et al. (2011). It was concluded that the blind bolted joints had large bearing capacity and the rotational deformation capacity was good. Meanwhile, for blind bolted joints in CFST structures, thickening the local steel tube wall with cushion plates or anchoring bolt ends was adopted to strengthen the hooping force, but large deformation phenomena such as obvious bulge of the column wall, pulling-out of the blind bolt, cracking between the end-plate and the steel beam flange may still occur because of the single steel tube. Therefore, there is more theoretical and practical significance to use blind bolts for double-skin tubular columns. Theoretical and experimental studies on blind bolted joints were conducted for hollow circular and square CFDST columns (Wang et al. 2019), and the studies showed that the joints had good hysteretic performance, ductility and energy dissipation capacity. Then, in this chapter, blind bolts were used to solve the problem of beam-to-column connection in solid CFDST structures. Since blind bolts can be embedded in concrete for solid CFDST columns with the section of inner circular-outer square, they play a better role in force transferring. Based on this idea, joint specimens were designed and

tested under low-frequency cyclic loading. This chapter uses quasi-static low-cycle reciprocating loading tests and the digital speckle correlation method (DSCM) to investigate seismic behaviors and force-transferring mechanism of blind bolted joints in CFDST structures. Meanwhile, a reasonable material constitutive relation and a failure criterion are selected to establish finite element models of specimens, which are used to compare with the test results. A tensile T-plate model is also proposed to calculate the ultimate bending capacity of blind bolted joints in this chapter. And the calculations are analyzed their reliabilities by comparisons between the test results and simulation results.

9.1 Specimens Design

Five blind bolted joint specimens with T-plates, named BBJ1, BBJ2, BBJ2D, BBJ3, BBJ4, and one through-bolted joint specimen named PBJ, were designed according to *Code for Design of Steel Structure* and *Code for Design of Steel-concrete Composite Structure* (Zhang et al. 2020b). The upper and lower flanges of the steel beam were connected with the T-plate webs by pre-tightening high strength bolts, the blind bolts were connected to the inside wall of the inner circular steel tube. The specific dimensions and details of specimens and the joint models are shown in Fig. 9.1. The total height of the column was 2070 mm and the total length of the beam was 3700 mm. The section dimension of the inner circular steel tube was $194 \times 6 \text{ mm}^2$, and the outer square steel tube was $280 \times 10 \text{ mm}^2$. Two kinds of H-shaped steel were used including $\text{H}346 \times 174 \times 6 \times 9 \text{ mm}^4$ and $\text{H}350 \times 175 \times 7 \times 11 \text{ mm}^4$. T-plates were cut from corresponding H-shaped steel, and two types of cross-section were $250 \times 200 \times 9 \times 14 \text{ mm}^4$ and $250 \times 200 \times 12 \times 20 \text{ mm}^4$. The type of both blind bolt and through-bolt was M20. The main test parameters of the specimens are shown in Table 9.1, where the axial compression ratio n was set at 0.27.

9.2 Material Properties

Concrete cubes with side length 150 mm were made when concrete was cast into the specimens, and axial compression tests were carried out according to the standard for test method of mechanical properties on ordinary concrete. The average compressive strength f_{cu} measured in two groups of concrete cubes was 60 MPa and the elastic modulus measured was 36142 MPa. Meanwhile, standard tensile specimens of the square steel tube, circular steel tube, T-plate, steel beam, and stiffening rib were cut from the same batch of steel. Three specimens were taken from each group and the average was taken as shown in Table 9.2.

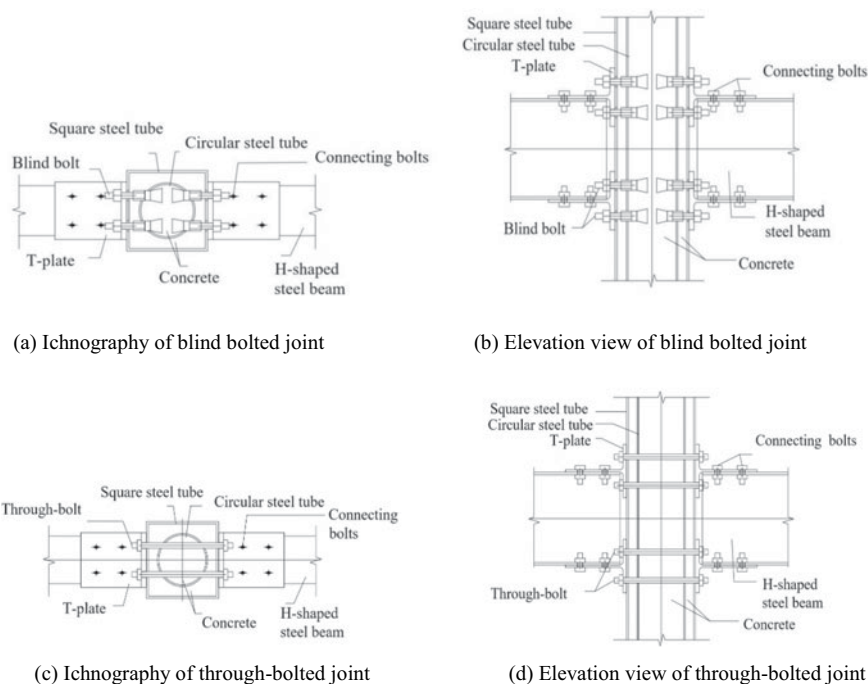


Fig. 9.1 Dimensions and details of specimens

9.3 Test Setup and Loading Scheme

9.3.1 Test Setup

The photo of the loading device is shown in Fig. 9.2. The test setup and loading system refer to Sect. 7.3.2. The load range of the horizontal actuator was ± 100 t, and the displacement range was ± 200 mm. Load-displacement mixed control mode was used in the test according to ATC-24 guidelines for cyclic testing.

9.3.2 Arrangement of Measurement Points

The same arrangement scheme was adopted for each specimen. Circumferential and vertical uniaxial strain gauges were arranged on the upper, middle and lower positions respectively, for inner and outer steel tube walls, as shown in Fig. 9.3a. Uniaxial strain gauges were arranged at the T-plate flange, T-plate web near the conjunction and bolts. Strain rosettes were arranged at the corner point and center in the panel zone, and there was a strain rosette at the central position of the stiffening

Table 9.1 Parameters of specimens

Specimen number	Outer tube $\square B \times t$	Inner tube $\Phi 2r_i \times t_i$	T-plate	Beam cross section	Rib number on each T-plate
BBJ1	$\square 280\text{mm} \times 10\text{mm}$	$\Phi 194\text{mm} \times 6\text{mm}$	$250\text{mm} \times 200\text{mm} \times 9\text{mm} \times 14\text{mm}$	$346\text{mm} \times 174\text{mm} \times 6\text{mm} \times 9\text{mm}$	No ribs
BBJ2	$\square 280\text{mm} \times 10\text{mm}$	$\Phi 194\text{mm} \times 6\text{mm}$	$250\text{mm} \times 200\text{mm} \times 9\text{mm} \times 14\text{mm}$	$346\text{mm} \times 174\text{mm} \times 6\text{mm} \times 9\text{mm}$	Single rib
BBJ2D	$\square 280\text{mm} \times 10\text{mm}$	$\Phi 194\text{mm} \times 6\text{mm}$	$250\text{mm} \times 200\text{mm} \times 9\text{mm} \times 14\text{mm}$	$346\text{mm} \times 174\text{mm} \times 6\text{mm} \times 9\text{mm}$	Double ribs
BBJ3	$\square 280\text{mm} \times 10\text{mm}$	$\Phi 194\text{mm} \times 6\text{mm}$	$250\text{mm} \times 200\text{mm} \times 12\text{mm} \times 20\text{mm}$	$346\text{mm} \times 174\text{mm} \times 6\text{mm} \times 9\text{mm}$	No ribs
BBJ4	$\square 280\text{mm} \times 10\text{mm}$	$\Phi 194\text{mm} \times 6\text{mm}$	$250\text{mm} \times 200\text{mm} \times 12\text{mm} \times 20\text{mm}$	$350\text{mm} \times 175\text{mm} \times 7\text{mm} \times 11\text{mm}$	No ribs
PBJ	$\square 280\text{mm} \times 10\text{mm}$	$\Phi 194\text{mm} \times 6\text{mm}$	$250\text{mm} \times 200\text{mm} \times 9\text{mm} \times 14\text{mm}$	$346\text{mm} \times 174\text{mm} \times 6\text{mm} \times 9\text{mm}$	Single rib

Table 9.2 Material properties of steel plates (MPa)

Steel	Square tube	Circular tube	T-shaped part		Steel beam		Rib
			$250 \times 200 \times 9 \times 14$	$250 \times 200 \times 12 \times 20$	$\text{H}346 \times 174 \times 6 \times 9$	$\text{H}350 \times 175 \times 7 \times 11$	
Yield strength	337.90	343.88	346.78	313.99	288.69	327.64	295.51
Ultimate strength	460.38	443.45	454.03	444.17	405.41	465.01	406.89
Elastic modulus	2.13×10^5	2.11×10^5	2.19×10^5	2.17×10^5	2.00×10^5	2.18×10^5	2.03×10^5

rib, as shown in Fig. 9.3b. Uniaxial strain gauges were arranged at the upper and lower steel beam flanges, where the points were 100 mm away from the T-plate webs and 10 mm away from the steel beam edges. At the same cross-section where strain gauges were located at the beam flange, there were strain gauges on the beam web at the central axis and the positions of 20 mm away from the beam flanges, as shown

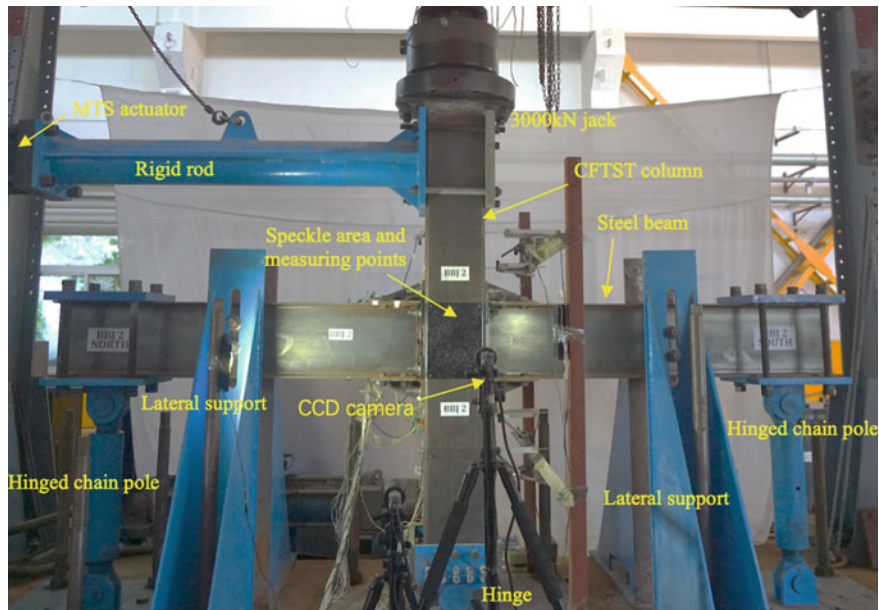


Fig. 9.2 Test setup of bolted joint

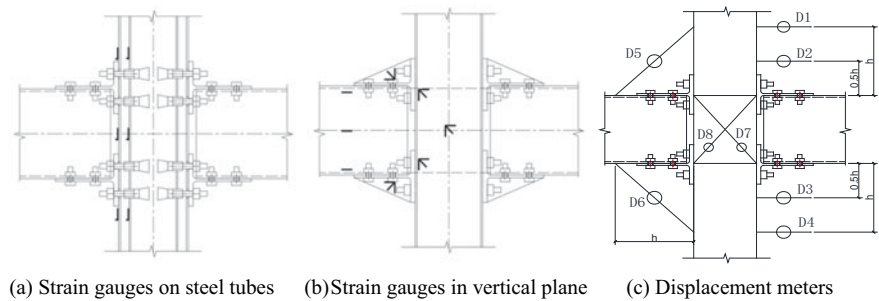


Fig. 9.3 Arrangement scheme of strain gauges and displacement meters

in Fig. 9.3b. The displacement meters are shown in Fig. 9.3c, and h stands for the steel beam height. D1, D2, D3, and D4 were used to measure the absolute horizontal displacement of the column. D5 and D6 were used to measure the beam-to-column rotation. D7 and D8 were used to measure shearing deformation in the panel zone.

9.4 Test Process and Phenomena

9.4.1 Failure Process

After the vertical axial force reached the specified value, the steel beams were fixed with the vertical supports, then the horizontal reciprocating load was applied by the horizontal actuator. The test phenomena of all specimens in the elastic stage were similar. Due to differences in structural constructions, the yielding order of the connection members was different.

- (1) Elastic stage: Each specimen was in the elastic working state during the load-controlling stage, and the steel strain changed linearly with the increase of load. When the load reached 130–160 kN, T-plates yielded firstly. T-plate webs yielded before flanges except for Specimen BBJ1 because the flange without stiffening rib of BBJ1 was thinner than other specimens. The strain of T-plate webs was between 1500–1800 $\mu\epsilon$, and the strain of the steel beam flange was between 500–900 $\mu\epsilon$, while the other strain values were small. With the increasing load, the load-displacement curves entered into a nonlinear stage. Friction noise was produced in the contact position between the steel beam flange and the T-plate web; and when the bolts reached to the anti-friction limit state, the contact position started to slide relatively.
- (2) Deformation stage: There was a gap of 0.5–1 mm between the T-plate flange and the outer tube at the displacement-controlling stage. The deformations of T-plates gradually became obvious and began to be visible. The deformation mode of T-plate depended on the number of stiffening ribs. The deformation form of T-plates in Specimens BBJ1, BBJ3, and BBJ4 without stiffening ribs is shown in Fig. 9.4a. The uniaxial bending deformation occurred on the T-plate flange, i.e., the upper and lower parts were closely attached to the outer tube wall, while there was a gap between the flange middle and the outer steel tube wall. The deformation form of the T-plate with a single rib in Specimens BBJ2 and PBJ is shown in Fig. 9.4b. The biaxial bending deformation of the T-plate occurred on both sides of the flange, i.e., the four corners were closely attached to the outer tube wall. The deformation form of T-plate with double ribs in Specimen BBJ2D is shown in Fig. 9.4c. The warpage deformation of the T-plate occurred on both sides of the flange, i.e., there were obvious gaps with the outer tube at the positions of stiffening ribs. In the early displacement loading stage, the gaps between the T-plate flange and the outer tube (see Fig. 9.4d) recovered during reverse loading. But with the load improvement, the flange deformation of T-plate increased rapidly for Specimens BBJ1, BBJ2, BBJ2D, and PBJ with 14 mm flange thickness; gaps were no longer recovered during reverse loading, as shown in Fig. 9.4e. When these four specimens were loaded to the third or fourth displacement cycle after the bending deformation of T-plate flange, the strain of steel beam flange started to exceed the material yield strain; steel began to peel as shown in Fig. 9.4f. While for the other two Specimens BBJ3, BBJ4



Fig. 9.4 Failure process photos

with flange thickness of 20 mm, the dislocation between the steel beam flange and T-plate web occurred in the latter stage of displacement loading; relative slippage and shearing deformation of connecting bolts occurred, as shown in Fig. 9.4g. Due to the increase of beam-to-column bending stiffness ratio for BBJ4, the maximum strain value of the lower steel beam flange was $1500 \mu\epsilon$, while the bending deformation of steel beam was not obvious during the whole loading process.



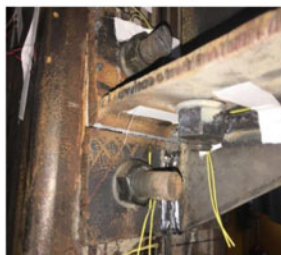
(j) Bending deformation of the steel beam web



(k) Bending deformation of the stiffening rib



(l) Cracking of the vertical weld



(m) Tearing of the T-plate flange (PBJ)

Fig. 9.4 (continued)

- (3) Failure stage: The connecting bolts of BBJ4 between the steel beam and the T-plate web were acted under larger internal forces, so the bolts of the lower flange of north steel beam were finally cut off, as shown in Fig. 9.4h. Then the testing of BBJ4 was stopped. Near the maximum lateral load of the other specimens, the steel beam flanges showed large bending deformation, as shown in Fig. 9.4i. The visible deformation occurred on the steel beam web of Specimens BBJ1, BBJ3 without stiffening ribs, as shown in Fig. 9.4j, because the T-plate without stiffening ribs made the larger beam-to-column rotation. Finally, the ribbed specimens BBJ2, BBJ2D, PBJ presented bending deformation on rib plates as shown in Fig. 9.4k. Especially, the vertical weld cracked for BBJ2D and PBJ due to the bending deformation reduction of T-plate flange caused by double ribs or through-bolts, as shown in Fig. 9.4l. Since the through-bolts of Specimen PBJ restricted the flange deformation of T-plate, the north T-plate flange near the web was also torn, as shown in Fig. 9.4m.

According to the failure process, it can be concluded that the T-plate flange was more prone to yield due to the tension from the T-plate web, so nearly all T-plates yielded as curved. But thicker flanges can increase bending stiffness and bearing capacity, which makes the steel beam form a plastic hinge earlier. The T-plate with a stiffening rib had larger overall stiffness. Stiffening ribs changed the state of T-plate webs from tension to bending. And ribs also restricted the relative flange-to-web rotation of T-plate, which led to a delay of the deformation of T-plate flange.

The deformation at beam ends of Specimens BBJ1, BBJ2, BBJ2D, BBJ3, and PBJ developed sufficiently and plastic hinges formed. But for BBJ4 with the larger beam-to-column bending stiffness ratio, the connection between the steel beam flange and the T-plate web needs to be strengthened due to the larger shear force. Through-bolts limited deformation of T-plate flange, so the brittle fractures occurred on the T-plate flange. The strain of the outer steel tube did not reach the yield strain, so no obvious deformation was observed for all outer steel tubular columns, but the strain distribution of the inner steel tube varied greatly with the bolt types. Neither did bolt shanks extend dramatically, nor did bolt nuts take off wires, so the blind bolted joint for the CFDST column is practicable and reliable to bear load.

9.4.2 Hysteresis Loops and Energy Dissipation Capacity

In this test, the P - Δ hysteresis loops of the column ends are shown in Fig. 9.5, the horizontal axis shows the horizontal displacement of the column top end and the vertical axis shows the corresponding horizontal load. The pushing direction (southward) of horizontal actuator is defined as a positive direction, and the pulling direction (northward) is defined as a negative position. It can be seen from Fig. 9.5 that shape of the hysteresis loop of each specimen changes from spindle shape to Z-shaped with the improvement of loading displacement; hysteresis loops show some shrinkage and extrusion due to the slippage phenomena from the early stage. Since T-plates and steel beams were in the elastic stage at the initial stage of loading, the hysteresis loop changed linearly. With the increase of displacement, T-plates and steel beams yielded successively, then there were gaps between T-plates and outer steel tubes. Relative slipping between the steel beam flange and the T-plate web can be observed, which led to the slippage property in hysteresis loops. After entering into the elastic-plastic stage, the elastic deformation restored quickly in the process of unloading and reverse reloading in each cycle, and the curves began to show inflection points. Due to the slipping and plastic deformation, each cyclic curve has a long horizontal stage, i.e., when the column end displacement backs to the original zero point until it increases reversely, the load has a little change. And then T-plates enter into the material strengthening stage, the load rises rapidly with the displacement increase. During this stage, Specimens BBJ1, BBJ2, BBJ3, BBJ2D, and PBJ presented yielding and bending of steel beams except BBJ4 because of its larger beam-to-column bending stiffness ratio. The maximum lateral load in each cycle increases slowly with the increase of displacement and the hysteresis loop shows good deformation capacity. The hysteresis loops of ribbed Specimens BBJ2, BBJ2D, PBJ descend suddenly during the failure stage. Due to the stress concentration, there were bending and cracks on stiffening ribs, and through-bolted specimen PBJ presented the tearing of T-plate flange.

Compared with Specimen BBJ1 without stiffening ribs, hysteresis loops of BBJ2, BBJ2D are fuller, indicating that they have better energy dissipation capacity. The maximum lateral load of Specimen BBJ3 with thicker T-plates doesn't increase

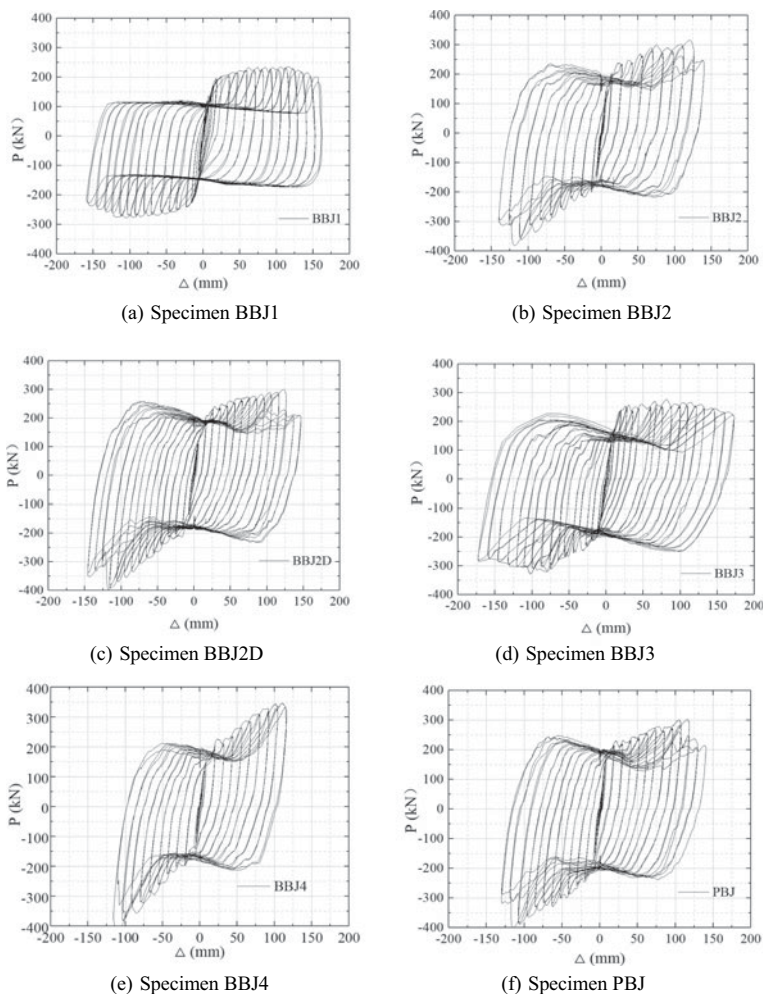
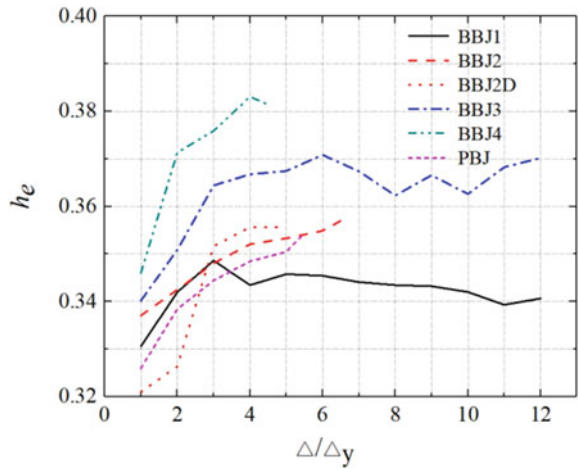


Fig. 9.5 Lateral load-displacement hysteresis loops

obviously than that of BBJ1, but the hysteresis loops of BBJ3 are wider and fuller. In this chapter, the equivalent viscous damping coefficient h_e is used to reflect the energy dissipation capacity of the specimens, as shown in Fig. 9.6. From the relationship between the equivalent viscous damping coefficient and each displacement level, it can be seen that h_e of each specimen is small at the initial stage and h_e increases gradually. The equivalent viscous damping coefficient of BBJ3 without stiffening ribs is always higher than that of BBJ1, indicating that the thickness increase of T-plate can improve the energy dissipation capacity. The equivalent viscous damping coefficient of blind-bolted specimen BBJ2 is higher than through-bolted specimen PBJ, and the brittle fracture that occurred on the T-plate flanges would not be beneficial for the

Fig. 9.6 Equivalent viscous damping coefficients versus numbers of cycles

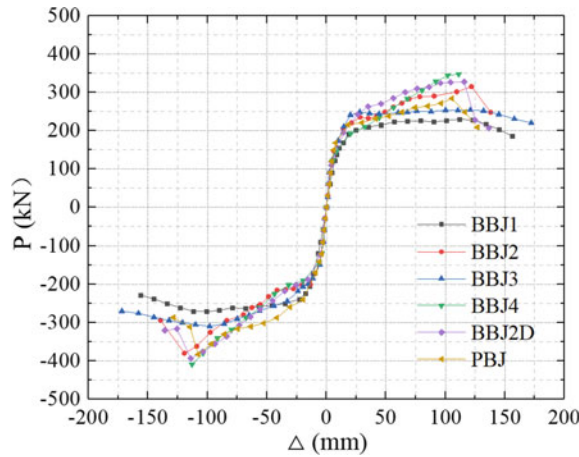


seismic resistance and energy dissipation of a structure. Therefore, the blind-bolted specimens exhibited high energy-dissipation capacities in the latter loading stage, which indicated that the blind-bolted joints achieved favorable seismic performance.

9.4.3 *P-Δ Skeleton Curves*

The load-displacement skeleton curves of all specimens are shown in Fig. 9.7. The skeleton curves have distinct elastic, elastic-plastic and plastic failure stages. All the specimens showed good deformability, and all the maximum displacement exceeded 100 mm. The lateral load at the column top can keep basically constant or increase

Fig. 9.7 Lateral load-displacement skeleton curves



slightly with displacement increase after the peak load. For Specimen BBJ4 with the larger beam-to-column bending stiffness ratio, though the connecting bolts between the steel beam flange and the T-plate web were cut off, the maximum load was still found to have the largest value. Due to the change of the force transferring path in ribbed specimens BBJ2, BBJ2D, the initial stiffness was larger than that of Specimen BBJ1 without stiffening ribs, and the maximum lateral load also increased obviously, but deformation capacity was relatively decreased. Compared with through-bolted joint PBJ, Specimen BBJ2 only substituted blind bolts to connect T-plates with the CFDST column. Synthesize the positive and negative results obtained by the pushing and pulling forces coming from the horizontal actuator, the blind bolted joint BBJ2 has the slightly smaller initial stiffness, while the maximum lateral load is almost the same as that of PBJ.

9.4.4 Bearing Capacity and Ductility

The corresponding load and displacement of the yield point, peak point and failure point in the pushing and pulling direction are shown in Table 9.3. Ductility is an index to reflect the capacity of a structure or a member to make full use of plastic deformation after yielding. Ductility coefficient is the displacement ratio of failure point over yield point. Because the bolted joint with T-plates is a typical semi-rigid connection form, it can be seen from the table, the displacement ductility coefficients of the specimens are between 3.21–11.80, so the blind bolted joint specimens of CFDST columns have good ductility, which meets the requirements of the codes for seismic design of buildings.

Table 9.3 Test results

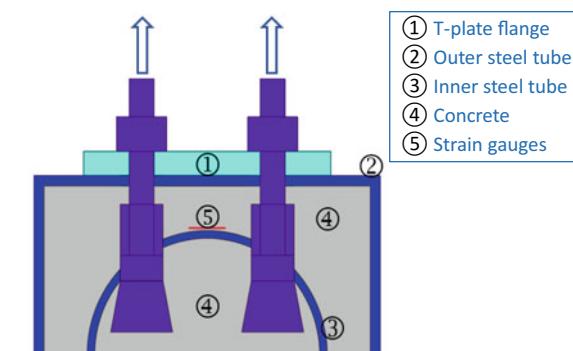
Specimen and direction		Yield point		Peak point		Failure point		Ductility coefficient
		P_y (kN)	Δ_y (mm)	P_m (kN)	Δ_m (mm)	P_u (kN)	Δ_u (mm)	
BBJ1	Push	162.75	12.68	230.41	112.03	195.85	149.66	11.80
	Pull	-226.08	-16.60	-270.11	-112.37	-229.59	-154.25	9.29
BBJ2	Push	221.27	21.49	315.20	122.01	267.92	133.35	6.21
	Pull	-211.91	-30.16	-379.93	-119.34	-322.94	-132.64	4.40
BBJ2D	Push	232.57	24.23	327.88	115.79	278.70	120.34	4.97
	Pull	-225.45	-38.24	-393.06	-114.12	-334.10	-122.93	3.21
BBJ3	Push	205.15	14.88	254.02	121.30	215.92	172.30	11.58
	Pull	-208.66	-20.76	-309.97	-98.17	-263.47	-172.57	8.31
BBJ4	Push	199.86	25.16	347.81	111.29	347.81	111.29	4.42
	Pull	-207.49	-34.81	-409.17	-112.99	-409.17	-112.99	3.25
PBJ	Push	217.49	22.45	284.36	105.54	241.71	118.20	5.27
	Pull	-259.28	-30.68	-383.02	-107.82	-325.57	-113.32	3.69

Bearing capacity of the bolted joint is the maximum lateral load applied on the column top herein, as shown in skeleton curves (Fig. 9.7). Thus, bearing capacity of BBJ2 with a single rib on each T-plate is 36% higher than that of the unribbed specimen BBJ1, and bearing capacity of BBJ2D with double ribs on each T-plate is 40% higher than that of BBJ1. The ultimate displacement of BBJ3 with T-plate flange thickness 20 mm is 172 mm, which is 13% higher than that of BBJ1 with T-plate flange thickness 14 mm. The bearing capacity is also increased by 13%. This attributes to bending strength improvement of the thicker T-plate flange. Since the T-plate is the main force transferring member in the beam-to-column connection, the T-plate with a thicker flange can bear higher bearing capacity and structural deformation after the yielding of the steel beam. When the beam-to-column bending stiffness ratio increases, the bearing capacity of BBJ4 is 34% higher than that of BBJ3 when connecting bolts failed for BBJ4. The average ductility coefficients of Specimens BBJ2, BBJ2D, PBJ with stiffening ribs are 5.30, 4.09, and 4.48 respectively. The ductility coefficients are much lower than that of specimens without stiffening ribs, but the bearing capacity is increased by 30–40%. It shows that the stiffening rib on T-plate can change the force transfer path, and significantly improve the bearing capacity. Compare Specimens BBJ3 and BBJ2 with BBJ1 respectively, the steel volume of the thickened T-plate flange in BBJ3 is more than twice that of one stiffening rib in BBJ2, but the bearing capacity of BBJ3, BBJ2 is improved by 13%, 36% respectively than that of BBJ1. It can be concluded that the effect of installing the stiffening rib is much better than thickening the T-plate flange. The bearing capacity of PBJ with through-bolts is close to that of BBJ2 with blind bolts, while the ductility of PBJ is less by 15%. In the failure stage, due to yielding of stiffening ribs or weld cracks, the curves of specimens with stiffening ribs show a sharp decline. Especially, the ultimate bending capacity of BBJ2D was not significantly higher than that of BBJ2 because the vertical weld between the stiffening rib and the T-plate flange cracked. It indicated that the welding quality could provide enough rigidity and strength to the T-plate, which could avoid the unexpected failure modes occurred in ribbed connections.

9.4.5 Strain Responses of Inner Steel Tubes

It is important to study the force transfer law between blind bolts and inner tubes for the design of connection. The tensile force from the steel beam can be transferred to the inner steel tube and the core concrete by bolts, while the compressive force is transferred to the joint core directly by T-plate flange itself. For the blind bolts in CFDST column, the pull-out model is shown in Fig. 9.8. In this test, circumferential and vertical strain gauges were arranged at the upper, middle, and lower points on the north side of the inner steel tube (see Fig. 9.3a), which were used to study the difference of strain change from blind bolted joints to through-bolted joints. The measured strain value of inner tubes is large for the pull-out model of blind bolts, and conversely, the obtained value is very small. Bending moment in the beam section

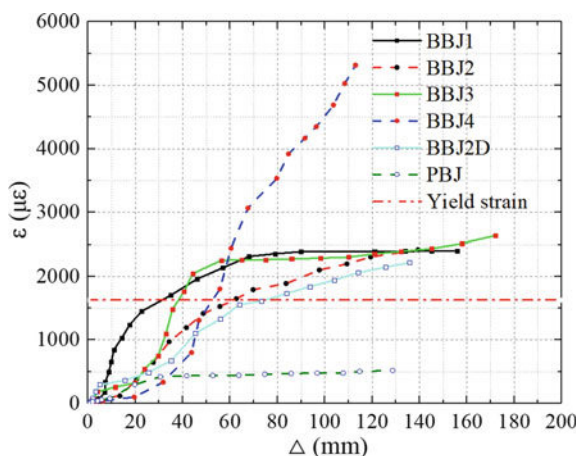
Fig. 9.8 Pull-out model of blind bolts



can be considered as the transformation of an internal tensile force and an internal compressive force on the upper and lower steel beam flanges respectively. Therefore, the blind bolts worked alternately with the horizontal pushing and pulling force at the column top.

Figure 9.9 shows the relationship between the pulling displacement at the column top and the vertical strain of lower point in inner steel tube where the blind bolts were transferring tensile forces. As shown in the figure, at the initial stage of loading, the specimens were in the elastic stage, and the strain of the inner steel tube changed a little with the load increase. As the load continued to increase, the flanges and webs of T-plates of each specimen yielded successively, and the strain gradually increased. For PBJ, the inner steel tube worked in a whole CFDST column by through-bolts for PBJ, so the inner steel tube was always in the elastic stage. The strain response is very different for blind bolted joints owing to the direct stress on the inner tube. The strain of inner steel tubes increases rapidly, and all exceeds yield strain at failure stage as shown in Fig. 9.9. It indicates that inner tubes in blind bolted joints produce

Fig. 9.9 Strain diagram of inner steel tube



strong anchoring force to guarantee the force transferring in the beam-to-column connection. For Specimens BBJ1, BBJ2, BBJ2D, and BBJ3, the strain of inner steel tubes is almost the same since the same type of steel beam produces similar tensile force through blind bolts. However, the bearing capacity of these four specimens with different stiffening ribs and flange thickness of T-plates was differentiated obviously. It can be concluded that different T-plates have a little effect on the bearing capacity when blind bolts are under tension, but the effect is obvious when T-plates are under compression. While for Specimen BBJ4 with the larger beam-to-column bending stiffness ratio than BBJ1, the strain of inner steel tube increased rapidly; meanwhile, the bearing capacity was much improved. Moreover, since inner steel tubes were embedded between the exterior and interior concrete, the stress concentration around the anchored head of blind bolts is not evident and inner tube wall continues to provide more resistance until the bending failure of the steel beam. It can be predicted that there was no excessive deformation on concrete and inner tubes. Therefore, the pull-out behavior of the blind bolt in CFDST column performs well owing to the section characteristics of the double steel tubes, blind bolted joints exhibit excellent structural internal force redistribution.

9.5 Finite Element Modeling

9.5.1 Establishment of Finite Element Model

FE simulation was carried out by the ABAQUS program to obtain more practical data to analyze the mechanical behavior of the blind bolted joint in a CFDST structure. The increment theory of constitutive relationship between stress and strain (σ - ε) was adopted to establish FE models. The stress-strain relation of steel and concrete refers to Sect. 6.6.1.

An 8-node linear reduced integral element C3D8R and a self-adaptive meshing method were adopted in the process of establishing a FE model. In order to speed up the simulation calculation, only half joint was established because the geometric distribution, boundary conditions and loading were symmetrical. The loading mode was controlled by unicyclic displacement. The boundary conditions and mesh generation of the finite element model are shown in Fig. 9.10.

9.5.2 Simulation of Failure Modes

The comparison of failure modes between test and FE analyses is shown in Fig. 9.11 respectively. Since shear failure of the connecting bolts occurred in the test specimen BBJ4, it can't be simulated in the FE modeling. Specimens PBJ and BBJ2 with single stiffening rib presented bending deformation on the stiffening rib plate as shown in

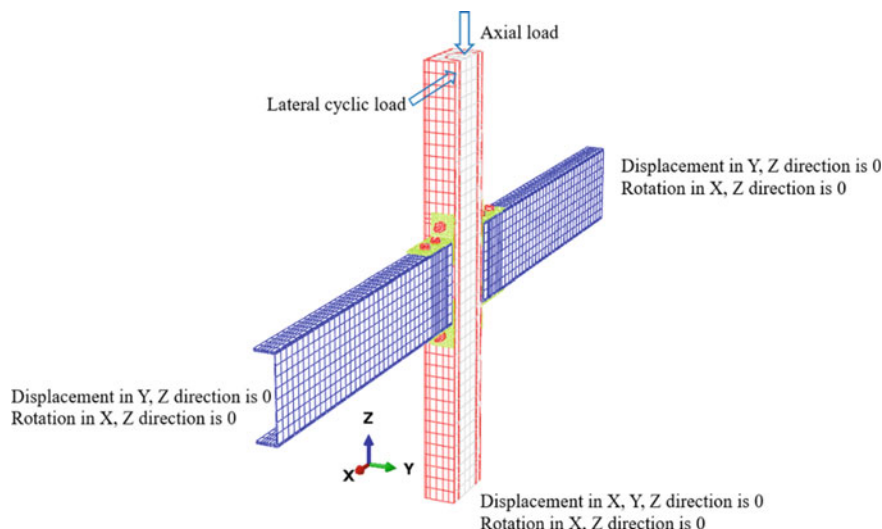


Fig. 9.10 Meshing, loading mode and boundary condition of FE simulation

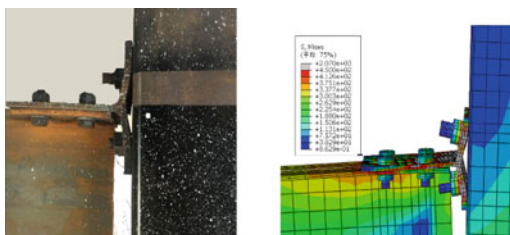
Fig. 9.11b. Due to the thicker T-plates flange of BBJ3, though the flange yielded, there was no larger deformation to produce a wider gap like the other bolted joints in the final stage, as shown in Fig. 9.11d. Since the through-bolts restricted the flange deformation of T-plate, the brittle fracture like the tearing of north T-plate flange near the T-plate web for Specimen PBJ is shown in Fig. 9.11e. In general, bolted joint to the CFDST column fails from the yielding of T-plate flange, so there appears a gap between the T-plate and the CFDST column; then plastic deformation at steel beam occurs, which is the same as the numerical prediction. The panel zone had no obvious deformation, and outer steel tubes did not yield, but the measured strain values of inner steel tubes were large and beyond the yield value for blind bolted joints.

9.5.3 Comparison of Ultimate Bending Capacity Between Test and FEA Results

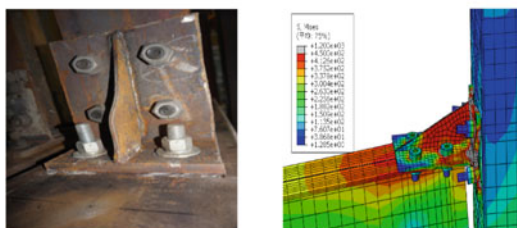
9.5.3.1 θ Measured by DSCM System

The DSCM system (see Sect. 7.6.1) is also applied in this test. The measuring points in the DSCM observation scheme for beam-to-column relative rotation are shown in Fig. 9.12. Three points are marked white on the specimens. Point 1 is located at the column wall, and the distance from this point to the lower steel beam flange is the beam height. Point 2 is located at the lower steel beam flange, and the distance to the column wall is also the beam height. Point 3 is at the intersection of the steel beam and the outer steel tube. The beam-to-column rotation at every moment can be

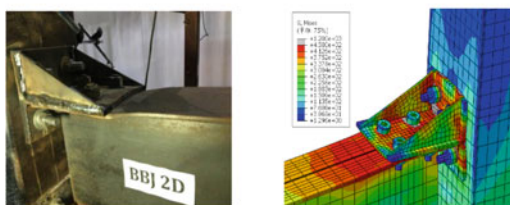
Fig. 9.11 Failure process of specimens



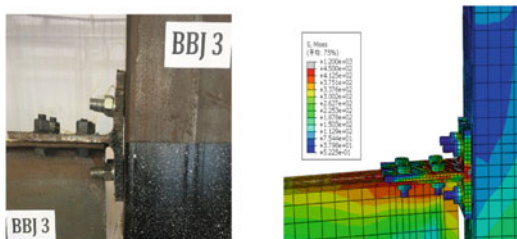
(a) Comparison of failure modes of BBJ1 between the test and the simulation



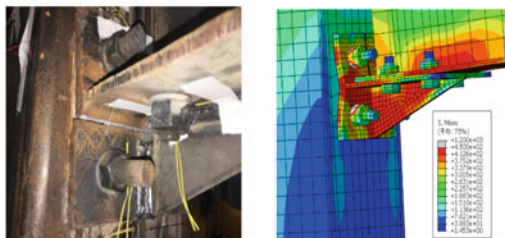
(b) Comparison of failure modes of BBJ2 between the test and the simulation



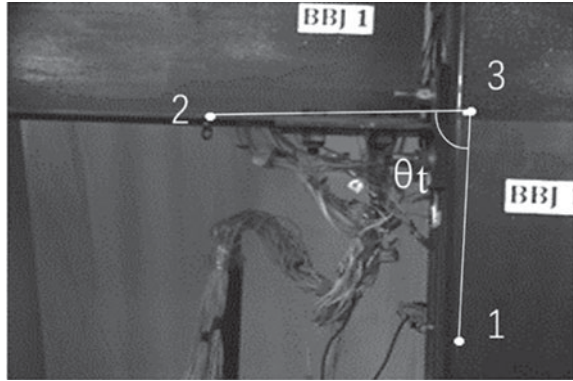
(c) Comparison of failure modes of BBJ2D between the test and the simulation



(d) Comparison of failure modes of BBJ3 between the test and the simulation



(e) Comparison of failure modes of PBJ between the test and the simulation

Fig. 9.12 Measuring points

calculated by the anti-triangle cosine formula. Then, the relative rotation θ between the beam and the column is

$$\theta = \theta_t - \theta_{t0} \quad (9.1)$$

where θ_t is the angle at time t as shown in Fig. 9.12, and θ_{t0} is the initial angle.

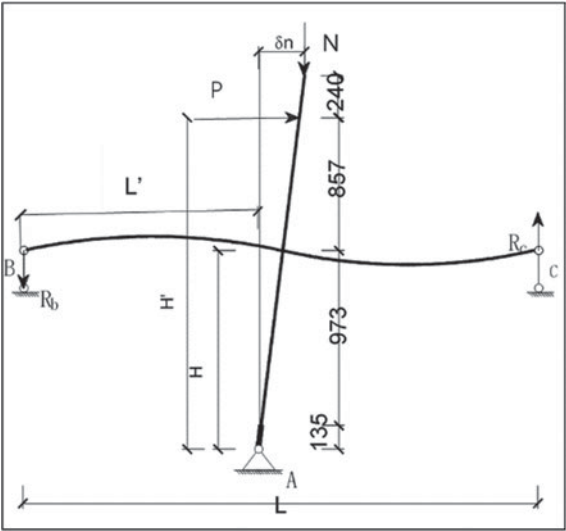
9.5.3.2 Calculation of Bending Moment at Beam End Section

The horizontal force was applied on the side of the column top, so the loading point was located at the center of the loading surface. Meanwhile, the center of the hinged support was 135 mm away from the column bottom. Then the simplified calculation diagram is shown in Fig. 9.13. The specimen was designed in bilateral symmetry, so it can be found that reactions at points B and C were equal in magnitude and opposite in direction. The bending moment (M) at the beam end section can be calculated using the following formula.

$$M = \frac{P \cdot H' + N \cdot \delta_n}{L} \cdot L' \quad (9.2)$$

where P is the horizontal force; N is the axial force on the column top; H' is the distance from the loading point of horizontal force to the hinged support; δ_n is the horizontal displacement of the column top; L is the beam length; L' is the length from the beam end to the column surface.

Fig. 9.13 Simplified force diagram



9.5.3.3 $M-\theta$ Hysteresis Loops

The bending moment(M)-rotation(θ) hysteresis loops obtained by the test and FE analysis are compared in Fig. 9.14. Hysteresis loops of all specimens change from shuttle-shaped to Z-shaped. During the displacement-controlled loading, T-plates of

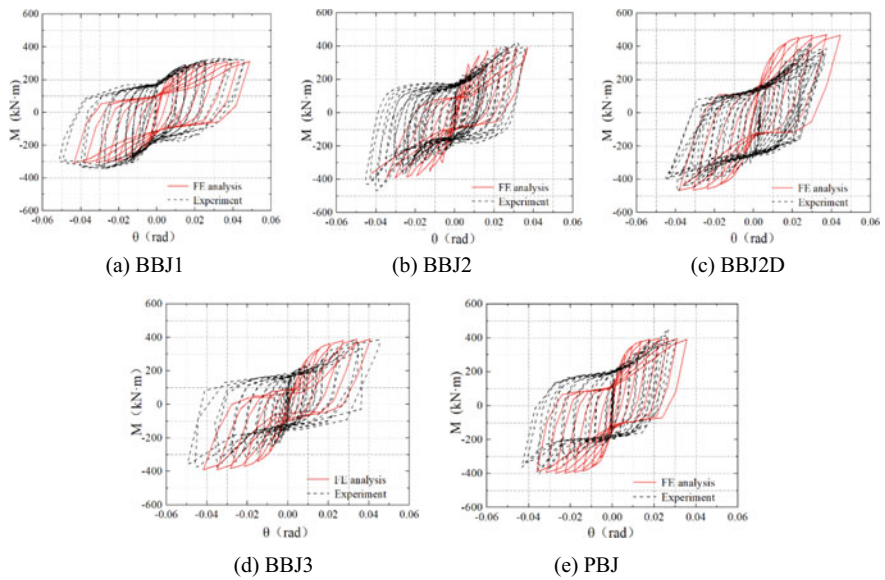


Fig. 9.14 Comparison of $M-\theta$ hysteresis loops between tests and FE analyses

each specimen had large deformation and the gaps with columns led to the slippage property in hysteresis loops. But the curves are still full, which verifies that the bolted joints in CFDST structures have good energy dissipation capacities. Each hysteresis loop of the bolted specimen has a horizontal strengthening stage after the elastic-plastic stage, and the joint stiffness begins to degenerate steadily in the late stage of loading. In particular, the blind bolted joint has the better deformability than the through-bolted joint in the later loading process. Though the hysteresis loops obtained by FE analysis present a phenomenon of more pinching than test results since the viscous effect of materials cannot be considered in the FE models, and the curves are still in good agreement with test data. Therefore, both test and FE analysis results show that the blind bolted joints for CFDST columns have better working performance including energy dissipation capacity and ductility.

9.5.4 Stress Responses of Steel Tubes and Bolts

In Sect. 9.5.5, strain responses of the inner steel tube was analyzed to study the difference of strain change from the blind bolted joint to the through-bolted joint. To compare with those results, the stress distribution of steel tubes together with the bolts is shown in Fig. 9.15 for Specimens BBJ2 and PBJ at the peak load. The stress of the outer tube remains below the yield value for both through-bolted joint and blind bolted joint, but the stress distribution of inner tubes is very different, which is consistent with the strain responses obtained by test measurements. The inner tube of the through-bolted joint PBJ is elastic at the peak load, while it enters into the plastic stage for the blind bolted joint BBJ2. Because the CFDST columns are firstly under axial compression, the stress value of steel tubes on the tension side especially adjacent to the blind bolts is lower than that of the compression side. Through-bolts connect T-plates with the whole CFDST column section to transfer the internal forces, so the stress distribution of each through-bolt is almost equivalent

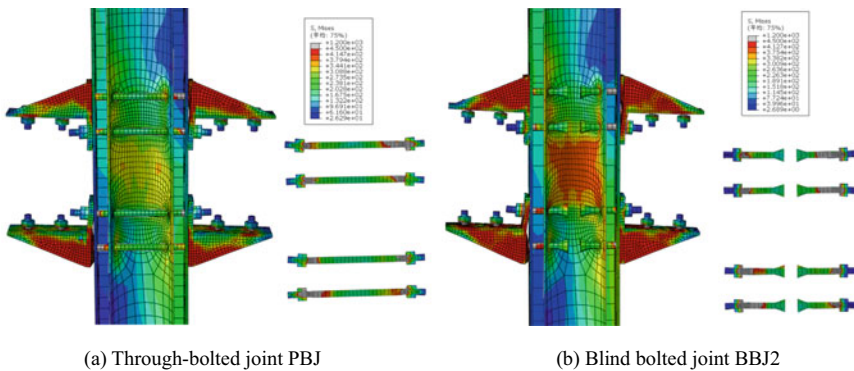


Fig. 9.15 Stress nephograms of steel tubes together with the bolts

without consideration of the difference between tension and compression, as shown in Fig. 9.15a. But in the case of the blind bolted joint, blind bolts are anchored to the inner wall of the circular steel tube, so the tensile force from the steel beam can be transferred to the inner steel tube and the core concrete by bolts, as shown in Fig. 9.15b. Through the analysis of stress responses, it can be concluded that the blind bolts obtain strong anchoring forces to guarantee the force transferring in the joint core. T-plates with high-strength bolts are the direct internal force-transferring members.

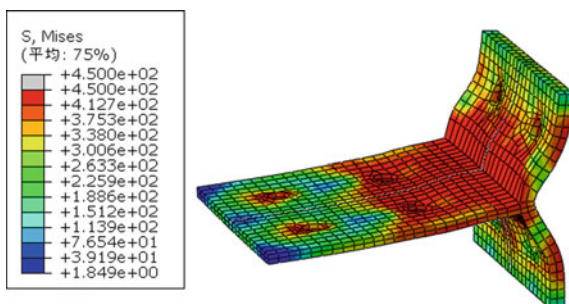
9.6 Calculation of Ultimate Bending Capacity

9.6.1 Establishment of a Tensile T-Plate Model

According to the aforementioned observation from tests and FE simulations, the blind bolted joint took good advantage of the sectional characteristics of the CFDST column and it had a better mechanical performance. T-plate is the direct force transferring member in the blind bolted joint. Bending moment at beam end is transformed into tension and compression in steel beam flanges. The tension is transferred from T-plate webs by connecting bolts to T-plate flanges, then blind bolts transferred internal force to the joint core, while the pressure is directly transferred to the outer tube through T-plate flanges. It is well known that the compressed T-plate is always working when the tensile T-plate meets the design requirements in a specimen. Therefore, the tensile T-plate and its structural performance determines the ultimate bending capacity of the joint. The specimen failed from the bending deformation of T-plate flange in the test. It is also verified that the tensile model of T-plate can be used to calculate the ultimate bending capacity. Figure 9.16 shows the stress nephogram of the T-plate nearly at the peak load. T-plate has entered the strain hardening stage and the yield stress zone would determine whether the steel beam could yield or bend.

The flanges thickness of T-plates is an important factor to affect its strength. Thinner flange of the T-plate makes its bending stiffness smaller, which leads to

Fig. 9.16 Stress nephogram of T-plate



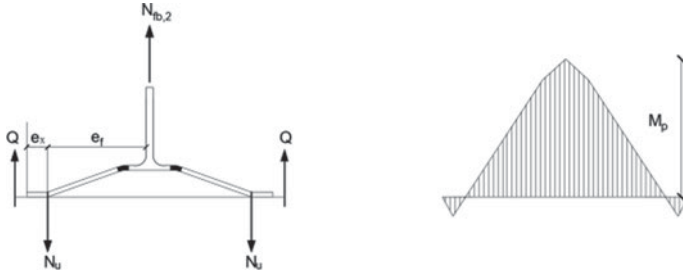


Fig. 9.17 Force diagram and bending moment diagram of the T-plate

untimely failure of the T-plate and bearing capacity of the steel beam cannot be fully played. Thicker flange of the T-plate may cause high-strength blind bolts to be pulled out. According to relative stiffness between bolts and T-plates, the ideal tensile failure mode (Liu et al. 2007) of T-plates is that yield lines will form at the flange-web junction. Figure 9.17 shows the external forces acting on T-plate and the bending moment diagram of its flange, where Q is the prying force at the T-plate edge.

The bending moment is transferred by T-plates directly, so when a plastic hinge occurs at the T-plate flange, ultimate bending capacity M_p can be obtained from Eurocode 3 (2005).

$$M_p = b_{ep} t_{ep}^2 f_{u,ep} / 4 \quad (9.3)$$

An equilibrium equation derived from the yield condition is as follows.

$$N_{b0} e_f - Q(e_f + e_x) = M_p \quad (9.4)$$

When bolt tension N_{b0} reaches the limit state N_u , the prying force is

$$Q = \frac{N_u e_f - M_p}{e_f + e_x} \quad (9.5)$$

According to equilibrium conditions, it can be deduced as follows.

$$N_{fb} = 2(N_u - Q) \quad (9.6)$$

When the T-plate fails, the corresponding ultimate bending capacity is expressed by

$$M_T = N_{fb}(h_b + t_{bf}) \quad (9.7)$$

The ultimate bending capacity M'_u provided by the T-plate can be calculated by

$$M'_u = \frac{(b_{ep}t_{ep}^2f_{u,ep} + 4N_ue_x)(h_b + t_{bf})}{e_f + e_x} \quad (9.8)$$

In these formulas, N_{b0} is the bolt tension; e_f is the distance from the bolt center to web edge of the T-plate; e_x is the distance from the bolt center to flange edge of the T-plate; N_{fb} is the maximum tensile force of T-plate; b_{ep} is the width of T-plate; t_{eb} is the flange thickness of T-plate; $f_{u,ep}$ is the ultimate strength of T-plate; h_b is the steel beam height; t_{ef} is the flange thickness of steel beam; t_{bf} is web thickness of the T-plate.

9.6.2 Working Mechanism of Stiffening Ribs

A T-plate with stiffening ribs can improve its strength and stiffness. Stiffening ribs can help transfer force between the flanges and webs and can also reduce prying force at the T-plate edge. According to the test phenomena and the FE analysis, there are three deformation characteristics of the T-plate with different stiffening ribs, as shown in Fig. 9.18. A uniaxial bending deformation occurred on the T-plate flange without stiffening ribs as shown in Fig. 9.18a. The deformation form of T-plate with a single stiffening rib is shown in Fig. 9.18b. The biaxial bending deformation occurred on the flange because of the concentrated forces coming from the middle stiffening rib and the T-plate web. The warping deformation form of T-plate with double stiffening ribs is shown in Fig. 9.18c. Besides, according to the stress nephogram, stiffening

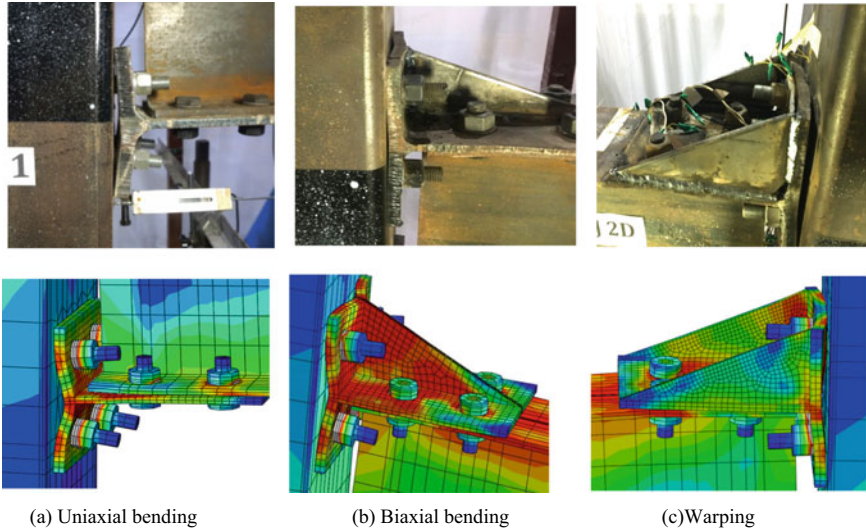


Fig. 9.18 Deformation characteristics of T-plates with different stiffening ribs

ribs changed the simple state of webs under tension and flanges under bending into both under complex bending. And stiffening ribs also restricted the relative flange-to-web rotation of T-plate, which led to a delay of the deformation of T-plate flange. Although different stiffening ribs influenced the deformation of T-plates, the final failure modes of blind bolted joints were similar as the beam flange bent significantly after the T-plate yielded.

Double-side fillet weld was adopted between the stiffening ribs and T-plates. The weld strength should be larger than the base metal to avoid tearing damage. The quality of welds will affect stress distribution and the failure mode, thus affect the bearing capacity of joints. Assume the tension P_w in the T-plate flange is distributed evenly along the fillet weld, so P_w transferred by fillet weld is

$$P_w = 2f_v^w l_{fv} h_{fv} + 2f_t^w l_{ft} h_{ft} \quad (9.9)$$

where h_{fv} and h_{ft} are respectively the height of shearing and tension fillet welds, l_{fv} and l_{ft} are respectively the length of shearing and tension fillet welds, f_v^w and f_t^w are the shear strength and tensile strength respectively.

The acting effect on the vertical side of the stiffening rib can be regarded as a uniform force, so the resultant force P_y is calculated by

$$P_y = k_y f_y h_s t_s \quad (9.10)$$

The coefficient k_y can be obtained by Samon's test (1964).

$$k_y = 1.39 - 2.20 \left(\frac{h_s}{l} \right) + 1.27 \left(\frac{h_s}{l} \right)^2 - 0.25 \left(\frac{h_s}{l} \right)^3 \quad (9.11)$$

where l is the length of stiffening rib; h_s is the height of stiffening rib; t_s is the stiffening rib thickness; f_y is the tensile strength of stiffening rib.

According to material strength in the test and the design value of fillet weld strength, it was concluded that all specimens satisfied the condition of $P_w \gg P_y$, i.e., the designed fillet weld does not crack before tearing and fatigue failure of the base material. However, the double fillet weld still cracked in BBJ2D, which indicates that actual field conditions and possible welding defects must be considered. It is better to take various measures to ensure weld quality and minimize heat-affected zone so that the mechanical properties of the stiffening rib can be fully utilized. If the double fillet weld can transfer the ultimate bending moment, the resultant force acting point is simplified as the stiffening rib bottom, so the contribution of bending moment M_l due to installing stiffening ribs on the T-plate is

$$M_l = P_y \left(\frac{h_b}{2} + t_{bf} \right) \quad (9.12)$$

Therefore, the ultimate bending capacity M_{cu} of blind bolted joints can be obtained by

$$M_{cu} = M'_u + x_l M_l \quad (9.13)$$

where x_l is the number of the stiffening rib.

9.6.3 Verification by Test Data and Numerical Results

The results M_{uc} calculated by the tensile model of T-plate are compared with test data, as shown in Table 9.4, where M_{ut} is the ultimate bending capacity from the test. It can be seen that the calculated ultimate bending capacity is very close to the test results. The mean error is 6.3%, and the maximum error is 9.3%, so it has verified the correction of the method of tensile T-plate model for blind bolted joints in the CFDST structure.

The failure modes and hysteresis loops obtained from the numerical simulation are confirmed to be in good agreement with test results in the foregoing discussion. The comparison of the ultimate bending capacity between simulations and tests (M_{um} and M_{ut}) are shown in Table 9.4. The mean error is 6.2%, and the maximum error is 10.5%. Therefore, the finite element models can be further used to analyze the influence factors of the bearing capacity. Fifteen numerical models were established based on the test specimen design to analyze the influences of T-plate thickness and stiffening ribs on the ultimate bending capacity of blind bolted joints, as shown in Table 9.4. In comparison of M_{um} and M_{uc} , the mean error is 2.2%, and the maximum error is 6.9%, so the calculation algorithm using the proposed tensile model of T-plate is also verified by FE simulation.

9.6.4 Analysis of Parameter Influence

The relationship curves between the improvement of the ultimate bending capacity and main parameters are shown in Fig. 9.19, where solid lines denote results about numerical simulation M_{um} , and dashed lines denote results about calculation M_{uc} . In response to the parameter influence curves, the change of ultimate bending capacity is obviously. In the theoretical analysis, the calculation value of ultimate bending capacity changes linearly with the increase of parameters, but the simulation results can show the interaction between different parameters. As can be seen in Fig. 9.19a, when the flange thickness of T-plates changes among 12, 14, 16, and 18 mm in FE analysis, the ultimate bending capacity is increased by 5.5, 19.8, and 23.2% in average in comparison with the corresponding joint with flange thickness 12 mm. As can be seen in Fig. 9.19b, the average ultimate bending capacity of joints with single stiffening rib is 28.0% higher than that of joints without stiffening ribs, while

Table 9.4 Calculation, test and simulation results of blind bolted joints

Specimens	T-plates	Stiffening rib	M_{ut} (kN·m)	M_{um} (kN·m)	M_{uc} (kN·m)	M_{uc}/M_{ut}	M_{um}/M_{ut}	M_{uc}/M_{um}
BBJ1-12	250mm × 200mm × 9mm × 12mm	None	/	297.345	302.324	/		101.7%
*BBJ1	250mm × 200mm × 9mm × 14mm		338.796	310.512	319.877	94.4%	91.7%	103.0%
BBJ1-16	250mm × 200mm × 9mm × 16mm		/	363.010	340.130	/		93.7%
BBJ1-18	250mm × 200mm × 9mm × 18mm		/	374.094	363.083	/		97.1%
BBJ2-12	250mm × 200mm × 9mm × 12mm	Singles	/	383.160	381.869	/		99.7%
*BBJ2	250mm × 200mm × 9mm × 14mm		440.471	394.410	399.422	90.7%	89.5%	101.0%
BBJ2-16	250mm × 200mm × 9mm × 16mm		/	424.415	419.675	/		98.9%
BBJ2-18	250mm × 200mm × 9mm × 18mm		/	436.884	442.628	/		101.1%
BBJ2D-12	250mm × 200mm × 9mm × 12mm	Double	/	465.873	461.414	/		99.0%
*BBJ2D	250mm × 200mm × 9mm × 14mm		449.786	468.730	478.967	106.5%	104.2%	102.2%
BBJ3-12	250mm × 200mm × 12mm × 12mm		/	309.873	309.744	/		99.9%
BBJ3-14	250mm × 200mm × 12mm × 14mm		/	330.550	327.728	/		99.1%
BBJ3-16	250mm × 200mm × 12mm × 16mm	None	/	364.160	348.477	/		95.7%
BBJ3-18	250mm × 200mm × 12mm × 18mm		/	373.033	371.995	/		99.7%
*BBJ3	250mm × 200mm × 12mm × 20mm		384.676	391.095	399.684	103.9%	101.7%	106.9%

Annotation: * represents test specimen

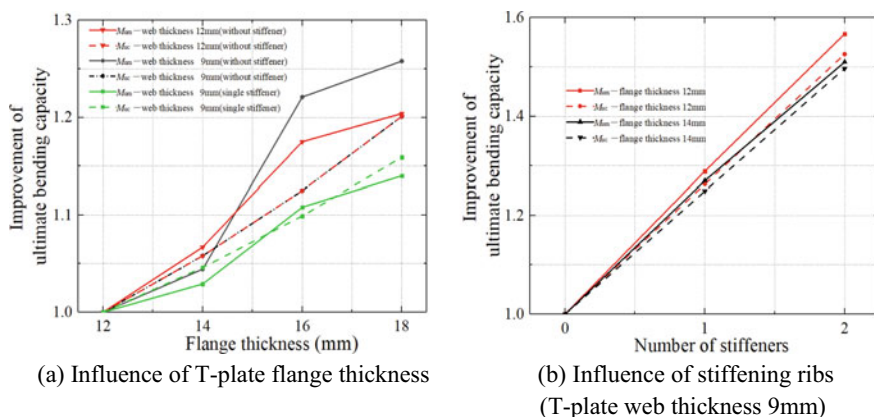


Fig. 9.19 Relationship between improvement of the ultimate bending capacity and main parameters

the average ultimate bending capacity of joints with double stiffening ribs is 53.9% higher, so FE analysis also illustrates that the ultimate bending capacity of Specimen BBJ2D with double stiffening ribs in the test was not obtained owing to the fracture of vertical welding seam between the stiffening rib and the T-plate flange. Comparing the data of simulation and calculation results in Table 9.4, the average ultimate bending capacity is increased by 2.7% when the web thickness of T-plates changes from 9 mm to 12 mm. Therefore, installing stiffening ribs on T-plates plays a leading role in improving the ultimate bending capacity, then thickening T-plate flange follows. The influence of thickening T-plate web is very weak.

On the other hand, installing stiffening ribs on T-plates or thickening T-plates can improve the bending stiffness of T-plate. According to parameter influence curves, the slope of straight lines is gradually reduced with the improvement of parameters, so the two dashed lines of calculation results about the web thickness of 9 and 12 mm coincide with each other in Fig. 9.19a. It can be concluded that the effect on improving the ultimate bending capacity slows down with the stiffness increase of T-plate. The main reason is that the T-plate will not determine the ultimate bending capacity when its bending stiffness is too large. Hence, the optimum strength matching of T-plates with blind bolts should be chosen to apply the equations derived by a tensile T-plate model in the calculation of ultimate bending capacity (Zhang et al. 2020a).

9.7 Summary

Bolted T-plate joints including five blind bolted joints and one through-bolted joint between concrete-filled double steel tubular (CFDST) columns and steel beams were tested under low-cyclic loading in this study. Finite element (FE) simulations were also carried out and verified by test results. Force-transferring mechanism was

analyzed, and the ultimate bending capacity of the blind bolted joints was calculated through the tensile T-plate model. The conclusions are drawn as follows.

- (1) The bolted joint to the CFDST column fails from the yielding of T-plate flange, so there appears a gap between the T-plate and the CFDST column, then plastic deformation at steel beam occurs, which is the same as the numerical prediction.
- (2) The hysteresis loops of blind bolted joints are full and the energy dissipation ability is strong. Test results show that the blind bolted joint has a slightly smaller initial stiffness than through-bolted joint, while the bearing capacity is almost the same and ductility is increased obviously. Strain responses of the inner tube in tests and stress responses in FE simulations show that blind bolted joints can give full play to the section characteristics of the double steel tubes and exhibit excellent structural internal force redistribution.
- (3) FE analysis results are in good agreement with the test data. The force-transferring mechanism of blind bolted joints is very different from the through-bolted joint. There are three deformation modes for T-plates with different numbers of stiffening ribs. T-plates with high-strength bolts are the direct internal force-transferring members. The blind bolted joint can give full use of the sectional characteristics of the double steel tubes and exhibit excellent structural internal force redistribution.
- (4) Installing stiffening ribs on T-plates plays a leading role in improving the ultimate bending capacity, then thickening T-plate flange follows. The influence of thickening T-plate web is very weak. The effect on improving the ultimate bending capacity slows down with the stiffness increase of T-plate.
- (5) The calculation algorithm using the proposed tensile model of T-plate is verified by test results and FE simulations. The differences of the calculation results comparing with the test and numerical results are within 9.3%. The optimum strength matching of T-plate with blind bolts should be chosen to apply the formula of calculating the ultimate bending capacity. The proposed analytical models in this chapter probably may provide a theoretical basis to realize refabricated construction of CFDST structures and the blind bolted joint could be a feasible choice for engineering practice.

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Chapter 10

Conclusions and Prospects



Abstract Some research conclusions about CFDST structure and the relevant study for the future work are discussed.

10.1 Conclusions

This monograph systematically introduced the technologies developed by authors for the field of CFST structures, with a particular emphasis on the column performance and joint construction of CFDST. The achieved outcomes will provide useful references for the future research in related areas.

Chapters 2 and 3 present the calculation methods for compressive strength and stiffness of reinforced CFSTs using the Unified Theory of CFST and the elastoplastic limit equilibrium method. Based on the Unified Theory of CFST, the proposed equivalent confinement coefficient is one important parameter to analyze axial behaviors of reinforced CFSTs. It has been verified that the equivalent confinement coefficient can calculate ultimate axial bearing capacity and composite stiffness of reinforced CFSTs. CFDST are testified to be the more applicable CFST structure by comparison of mechanical properties and steel ratio. And with consideration of the connection to frame beams, the cross-section with inner circular-outer square tubes is considered as more ideal one among all the various sections of reinforced CFSTs.

Chapters 4–9 describe the structural analysis for several types of connections to CFDST columns. A ring beam joint with the discontinuous outer tube is applied to connect RC beams, while the external diaphragm joint, vertical stiffener joint, anchored web joint, and blind bolted joint are applied to connect steel beams. To accomplish these studies, each research was generally divided into four phases for every connection in Chaps. 5–9: a prototype joint design, an experimental study on several specimens, a finite element simulation, and a data analysis and results comparison. In the first phase, joint specimens were designed respectively to bear the load required for tests and satisfy the corresponding Chinese codes and other standards. In the second phase of researches, some specimens with different parameters were fabricated and tested with a quasi-static loading method in a prototype frame. Strain gauges were pasted in some crucial positions to measure strains of steel, thus

load-strain curves were obtained and strain analysis of steel tubes can be conducted. The failure process and final failure mode of each specimen were recorded by taking photos. And in the loading process, relevant data including load applied to specimens and displacement were collected. Especially, in mechanical analysis of the external diaphragm joint and the blind bolted joint, the DSCM method was adopted to measure beam-to-column rotation. The third phase of researches was devoted to an inelastic finite element analysis of several joint models with different parameters. A reasonable material constitutive relation and a failure criterion were selected to establish finite element models of specimens. Then, whole loading processes were simulated. Failure modes, lots of relevant data and stress nephograms in simulation analyses were verified with test results. The last phase was data processing. The main parameters selected in these studies included: diameter-to-thickness ratio of steel tubes, axial compression ratio, structural form, beam-to-column linear stiffness ratio and so on. The analysis of seismic behaviors included failure modes, hysteresis loops, skeleton curves, ultimate bearing capacity, ductility, energy dissipation capacity, initial stiffness and stiffness degradation. Besides, some analytical models were established to predict and evaluate the ultimate bearing capacity, and calculation formulas were testified their accuracy by comparing with test results and FE results. The proposed analytical models may provide a theoretical basis for a promising application of the CFDST structure.

10.2 Prospects

In this end, recommendations for future research directions are given below:

The axial compressive strength and stiffness of CFDST columns can be calculated with a satisfactory precision by the proposed equivalent confinement coefficient, and it can be predicted that this equivalent confinement coefficient could be applied in the analysis of composite shear and bending performance. Then, a large number of tests and analyses should be carried out to verify the calculation of composite shear and bending resistance for CFDST columns.

Although several types of applicable frame joints to CFDST columns were described about constructional details and mechanical behaviors, it is essential to carry out more tests and corresponding FE analyses to select appropriate beam-to-column connection for various CFDST columns. CFDST has broad prospects and potentials in seismic design and practical application. Under the action of seismic load, the real structural system is subjected to extremely complex forces. The influence of floor slab on the spatial frame joint, and the different mechanical properties from the interior to exterior joints are all essential research focuses that need to be further studied in the future. It is advisable for lots of scholars to conduct in-depth researches on seismic performances of CFDST frames. Using FE analysis software or case tests, CFDST frame structure models composed of different beam-to-column connections can be evaluated by the ductility, energy dissipation capacity and deformation recovery performance, etc. to verify the excellent seismic-resisting performance of CFDST structures.