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Structural Performance and Evaluation of Square Steel Tube Encased Concrete Columns Confined by Circular Thin Steel Tube

張,弛

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Doctoral Dissertation

Structural Performance and Evaluation of Square Steel Tube Encased Concrete Columns Confined by Circular Thin Steel Tube (円形薄肉鋼管で横補強した正方形鋼管内蔵コ ンクリート柱の構造性能及び耐力評価に関す る研究)

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Graduate School of Engineering Kobe University ZHANG CHI 張 弛

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CHAPTER ONE Introduction

1.1 Backgrounds

Earthquake, as one of the most destructive natural disasters, has consistently exerted profound and widespread effects on human society and civilization. It not only results in significant material losses but also impacts people's lives, economies, and infrastructure. In recent decades, numerous mega-earthquakes such as 1995 Kobe earthquake, the 2008 Wenchuan earthquake, and the 2011 Great Tohoku earthquake have caused serious damage to numerous buildings and bridges as shown in Fig. 1.1. These disasters have resulted in tens of thousands of fatalities and left millions of people homeless [1.1].

After numerous devastating earthquakes, many countries and organizations have devoted substantial efforts to estimate the earthquake consequences and work on strategies to reduce their impacts. Following the 1978 Miyagi earthquake, Japan made its largest revision to the "Building Standards Act (New Earthquake Resistant Building Standard Amendment.)" in 1981[1.2], in which the new two phases design for earthquakes was added to ensure safety against severe earthquakes. The performance goal of the Chinese seismic design code (GB 50011-2001) [1.3] has been "no damage at minor earthquakes, repairable damage under moderate earthquakes, and no collapse under severe earthquakes", and achieve the above three levels of goals through two-stage design: the bearing capacity verification stage and elastic-plastic deformation stage. The IBC-2003 building specification in the United States [1.4] considered two levels of seismic action for structures, "design earthquake" and "maximum consideration earthquake", to achieve the seismic target of the structure.

In order to meet the objectives required in current seismic design, reinforced concrete (RC) structures have been widely used in construction projects around the world due to their cost-effectiveness, versatility, and ease of construction. However, they also have some obvious shortcomings when hit by stronger earthquakes than anticipation. At first, RC structures have a relatively high self-weight [1.9], which results in large inertia earthquake force and hence is a drawback. Secondly, due to the inherent softening property of concrete, concrete structures usually have limited ability to deform and/or absorb or dissipate the energy by ground motions.



Fig. 1-1 Damaged concrete building in major earthquakes [[1.9]

To overcome the disadvantages inherent in concrete structures, during the recent decades, there has been a growing interest in concrete-steel composite structures, which combine steel and concrete elements to enhance the seismic capacity of building structures, providing greater ductility, strength, and overall stability. Consequently, the composite structural components have found wide applications in building structures and infrastructure, including bridge piers, high-rise buildings, logistics warehouses, and other engineering buildings that must withstand substantial loads and under complex stress conditions.

The steel-concrete composite structures encompass various types of component forms, such as Steel-Reinforced Concrete (SRC) structures, Concrete-Filled Steel Tube (CFST) structures, Steel-Encased Concrete (SC) structures, among others.

A SRC component is generally composed of a cruciform or H-shaped steel element and surrounding concrete reinforced by longitudinal rebars. SRC components usually have larger load-bearing capacity and greater seismic resistance than RC Components besides effectively enhancing structural stiffness and minimizing lateral displacement [1.10]. However, the use of SRC columns in complex commercial buildings has been limited recently [1.11] due to the

complexities involved, such as intricate procedures for concrete pouring and reinforcement installation, potentially leading to increased labor and time costs.

Structurally benefitting from the composite interaction between the infilled concrete and outer steel tubes, CFST columns with enhanced strength and ductility have been increasingly utilized as gravity-sustaining components especially in high-rising buildings [1.12]. Besides, as compared with conventional SRC columns, the CFST columns could save construction steps, offer excellent seismic performance, and possess sound capability to dissipate energy induced by strong earthquakes. Nonetheless, CFST columns have their own drawbacks, including the relatively complex joint connections between columns and beams, a reduced lateral confinement effect on concrete due to the outer steel tube also bearing vertical stress, and the necessary fire resistance measures.

Among various kinds of building structures, logistics warehouses as shown in **Fig. 1-2** are indispensable and crucial components in modern supply chain of things. They play a vital role in providing essential support for the storage, handling, distribution, and management of industrial and agricultural products. The structures suitable for logistics warehouses need to meet a wide variety of specific requirements, including optimized floor layouts, maximizing height and space utilization, durability to withstand frequent loading and unloading, ensuring safety, eco-friendly design, and the necessary flexibility to adapt to evolving logistics needs.

In meeting these requirements for logistics warehouses, the selection of structural components is crucial. Concrete-steel composite columns is an ideal choice for such applications [1.15]. In the design of logistics warehouses, the use of concrete steel composite columns can create larger open spaces without the need for excessive internal support structures, which maximizes storage capacity and operational efficiency within the facility. Furthermore, the design of concrete-steel composite columns are engineered to withstand heavy equipment, automated material handling systems, and the vertical stacking of goods, ensuring the long-term stability and safety of the warehouse environment.

To overcome the difficulties and problems inherent in conventional SRC and CFST columns and to further promote the application of composite structures to the logistic warehouses located in the regions with high seismicity, a new type of steel-composite composite component is desirable.



Fig.1-2 Logistics warehouse [1.13-1.14]

1.2 Previous studies on concrete-steel composite columns

For the last several decades, SRC columns, CFST columns, CFST columns with steel bar (CFST-B) [1.16] or steel section embedded (CFST-S) [1.17], SC columns and other concrete steel composite columns [1.18] have been developed and studied.

As aforementioned, conventional SRC columns feature an encasing steel form (cruciform or Hshape) and longitudinal reinforcement into concrete that can effectively prevent local buckling of the inner steel forms [1.19], enhance ductility of the columns, and ensure efficient fire protection. Currently well-used SRC columns usually include welded or directly rolled steel plates formed into different cross-sectional shapes such as L-shape, H-shape and cruciform. In recent years, the seismic performance of some special shaped SRC columns and the issue of beam-column connection problems are being actively discussed by many scholars:

Nishimura et al. [1.20] clarified flexural behavior of SRC columns with T-shaped steel experimentally and theoretically, and the effects of the applied axial load and the loading angle to the principal axis of the reinforced concrete column section were studied. According to their experimental results, hysteretic response of SRC columns with T-shaped steel showed large energy dissipating capacity up to inter-story drift angle of 2% regardless of the applied axial load and loading angle, and the ultimate flexural strength of SRC columns was affected by the amount of steel and reinforcing bars placed at the flexural tension side. Jiang et al.[1.21] conducted numerical simulation on the mega SRC columns with different cross-section type of encased steel and steel ratio, and established nonlinear three-dimensional finite element and fiber element models respectively to simulate the inelastic behavior of the mega SRC columns.

Zhang et al. [1.22] studied a new joint design approach focusing on the seismic behavior of SRC special-shaped columns and RC beam-SRC column joints with a controlled stirrup spacing. According to their test results, the specimen with profiled steel was found to have better energy dissipation capacity. Wang et al. [1.23] investigated seismic performance of damaged SRC columns before/after repairing.

In order to tackle the intricacies of stirrups placed in SRC columns, the application of CFST columns, particularly concrete filled double skinned steel tubular (CFDST) composite columns, have gained more and more attention. Patel and Purohit [1.24] carried out an experimental investigation on the structural performance of ten square CFDST composite columns (as depicted in **Fig. 1.3**) under axial-flexural loading. In parallel to their experimental work, parametric study was also performed to confirm effects of the shape or orientation of the encased hollow steel tubes. The test results showed that CFDST columns with outer and inner square tubes under axial load performed well in terms of ultimate load capacity, confinement effect, ductility and displacement behavior, and that the shape and orientation of inner square steel tube did not have detrimental effect on axial strength of the CFDST columns.

Mou et al. [1.25] also researched the ultimate behavior and post-local buckling failures of circular CFST beam-columns, and experimentally verified that the CFST column specimens with a large D/t ratio of outer steel tube tended to trigger highly concentrated local buckling at near column bottom.

Meanwhile, the SC columns have recently found increasing applications particularly in meddle and high-rise buildings. The SC columns fully utilize the advantages of SRC columns and do not need the longitudinal rebars and transverse shear reinforcement, simplifying the production and construction process with greater efficiency. Moreover, they can enhance the efficiency of indoor space utilization, which is particularly advantageous for the buildings such as logistics warehouses that demand the maximization of available space.



Fig. 1-3 Geometrical details of CFDST specimens [1-24]

Kuratomi et al [1.26-1.29] have proposed a new type of square SC column (as depicted in **Fig. 1.4**), consisting of cross-shaped steel or H-shaped steel and concrete confined by square thin steel tubes. As displayed in Fig. 1.3, the square SC columns have neither longitudinal rebars nor transverse shear reinforcement, and the outer thin steel tube is only used to confine concrete aiming further enhancement of ductility of the columns. Based on the test results, they verified sound mechanical property and high ductility of this new type of square SC columns even under high axial load. The constitutive law of concrete and the evaluation method of concrete confinement effect, which depended on the proportion of internal steel ratio, were also examined.

Soliman et al. [1.30] assessed experimentally the current methods and codes for evaluating the ultimate load capacity of concrete encased steel short columns, while Lelkes et al. [1.31] made a theoretical analysis of the second-order theory at SC columns. Zhou et al. [1.32-1.33] studied the seismic behaviors of the circular and square tubed SC columns with inner I-shaped steel. **Fig. 1.5** shows the outline of this type of SC columns.



Fig. 1- 4 Typical cross-section of square SC columns [1.26-1.29]

The test results have indicated that the specimens confined by circular steel tubes exhibited higher lateral load strength, displacement ductility, and greater energy dissipation ability than common SRC short columns due to the effective confinement of circular steel tube to the core concrete.

From the research findings described above, it can be seen that the seismic performance of SC column is closely related to the confinement effect of the outer steel tube. Sun et al. [1.34-1.35] conducted axial compression tests on high-strength concrete short columns confined by circular and square steel tubes with varying thicknesses, and proposed the stress-strain curve models for confined concrete based on the experimental results. The proposed stress-strain curve models were applied to predict the flexural behavior of confined concrete columns under axial load and bending moment with moment gradient (shear), and a semi-empirical formula to estimate the flexural strength of confined concrete columns was proposed.



Fig. 1- 5 Concept of the Tubed SRC columns [1.32-1.33]

1.3 Research objectives

Based on the above-mentioned backgrounds and literature review, one can see that while there are several studies conducted to investigate structural behavior of SC columns, most of the previous studies placed emphasis on the square SC columns with I-shaped or cruciform steel encased. Furthermore, considering the fact that square steel tubes have much higher buckling resistance tubes than I-shaped and/or cruciform steels, as a new type of confined SC columns

suitable for logistics warehoused, this paper proposes square steel tube encased circular concrete columns confined by thin steel tubes. The proposed circular SC columns can make full use of the sound confinement effect by the outer circular steel tubes, which can also act as form for the columns. However, to promote the application of the proposed circular SC columns, several important issues still need to be addressed as described below:

- The effect of the grades of encased square steel tubes (FA rank, FB rank, and FC rank) on overall seismic performance of the proposed SC columns: On one hand, the overall structural performance of the proposed circular SC columns is dependent upon the grades of the encased inner steel tubes and the confinement effect by the outer thin steel tubes. On the other hand, the closer to FA rank the inner steel tubes, the worse the cost performance. To make more economical and reliable design of the proposed SC columns, it is necessary to clarify the influence of grades of the inner square steel tubes on the seismic performance of SC columns, and obtain basic data on the impact of the steel grades.
- 2) The influence of infilling concrete into the inner square steel tube on seismic performance of the proposed SC columns: It is apparent that without concrete being infilled into the inner steel tubes can save construction time and upgrade cost performance. However, if the grade of the inner square steel tubes is not high enough, the inner square steel tubes might prematurely buckled inwards, significantly reducing the ultimate capacities of the proposed SC columns. Therefore, to find the optimum grade of the inner steel tubes, information on the effect of the presence of infilling concrete is indispensable.
- 3) Quantitative evaluation of the confinement effect by circular thin steel tubes: In the proposed SC columns, in addition to the thin steel tube fabricated by welding, a new type of thin steel tube will be proposed. This new type of thin steel tube is fabricated by at first bending a flat steel plate with targeted thickness into semi-circular plate with connection flange at both ends of the semi-circular plate, and then clamping two plates by high-strength bolts and nuts along the height of the circular tube. The new type of circular steel tube is referred to as bolted tube in this paper. The bolted thin steel tube is developed to avoid potential rupture of the connection portion observed in the welded thin steel tubes. The bolded thin steel tube also has some advantages over the welded steel tube such as simplification of the assembly and disassembly, which enables structural engineers to easily observe the damage of concrete after suffering earthquake events. However, the

discontinuity of the bolted steel tube inevitably may sacrifice some confinement effect. Therefore, to rationally evaluate the structural capacities of the proposed circular SC columns, the confinement effect by the bolted steel tube needs to be confirmed and further compared with that by the welded thin steel tube.

4) Development of an evaluation method for the seismic capacities of the proposed SC columns: Although numerous seismic evaluation methods have been proposed for SRC components, it is not clear if these methods can be applied to the proposed circular SC columns. Particularly, it is not clear how we should model the encased square steel tube when conducting flexural analysis of the proposed SC columns.

Objectives of this doctor thesis are, 1) to obtain more experimental information on structural performance of the square steel tube encased concrete columns confined by circular thin steel tubes, 2) to investigate the effects of the grades of square encased steel tubes (FA rank, FB rank, and FC rank) on structural performance of the proposed SC columns, 3) to study the influence of the infilling concrete within the inner steel tube, 4) to compare and analyze the confinement effect of circular thin steel tubes with different thicknesses and joint methods, and 5) to find the reasonable design equation for assessing ultimate flexural strength of the proposed SC column section by comparing the experimental results with the calculated ones by representative code-prescribed equations, and to propose an numerical analysis method to evaluate the overall structural performance of the proposed SC columns.

1.4 Format of thesis

This dissertation consists of six chapters, and the key points of each chapter are summarized as below:

Chapter One introduces backgrounds of this dissertation, reviews previous researches in the literature, and describes research objectives of this study.

Chapter Two explores the axial performance of the circular SC short columns through axial compression tests. The SC short columns were composed of a FA rank square inner steel tube and an outer circular thin steel tube. The primary objectives of this chapter are, 1) to compare

the effect of infilling concrete in the inner steel tube on the axial behaviors of the SC short columns and the impact of local buckling of the inner steel tube, 2) to compare the confinement effect of outer circular thin steel tubes with different joint methods, 3) to study the influence of the outer-diameter-to-thickness ratio of circular thin steel tube on the strength enhancement ratio of confined concrete. Ten short columns were made and tested under axial loading. The experimental variables included the infilling concrete into inner steel tube, the joint method of outer circular thin steel tube (welded steel tube and bolted steel tube), and the thickness of outer steel tube (1.6mm and 2.3mm, outer-diameter-to-thickness ratio was 111.4 and 78.1, respectively).

Chapter Three is intended to obtain fundamental data on structural performance of the proposed circular SC columns confined by welded thin steel tubes with outer diameter-to-thickness ratio of about 189. The primary research targets are the impact of different grades of inner square steel tubes on the structural performance of the SC columns, the influence of the presence of infilled concrete within the encased steel tube, and verification of the confinement effect by the welded circular thin steel tubes. To these ends, a total of thirteen circular SC columns with diameter of about 300 mm were made and tested under reversed cyclic loading and constant axial compression to investigate the structural performance, ultimate strength, and defamation capacity. The experimental variables included the axial load ratio (0, 0.15, 0.25), the grades of inner square steel tubes (FA rank, FB rank and FC rank), the infilling concrete into the inner steel tube or not, and the presence or absence of main longitudinal rebars.

Chapter Four aims to experimentally verify effectiveness of the bolted circular thin steel tube on the cyclic performance of the proposed SC columns through comparing their cyclic behavior with those of the SC columns confined by the welded circular thin steel tube. Based on the observations made in chapter three, six circular concrete columns were made and tested under reversed cyclic lateral force and constant axial compression. All the specimens were confined by the bolted thin steel tubes. The primary experimental variables included the axial load ratio (0.15, 0.25), the grades of inner square steel tubes (FB rank and FC rank), the infilling concrete into the inner steel tube, and the thickness of the bolted circular thin steel tubes.

Chapter Five deals with the calculation method of the ultimate flexural strength of the proposed SC column sections as well as numerical analysis method of the overall structural performance of the proposed SC columns. To predict the ultimate flexural strength of the proposed SC

column sections, the design equations recommended in the AIJ standard for SRC columns and the AIJ design guideline for CFT columns were adopted. To take the confinement effect by the outer steel tubes into account, the compressive strength of concrete confined by the bolted steel tube will be evaluated by modifying the model proposed by Sun et al for the confinement by the welded thine steel tube defined. In addition, the core points of numerical analysis method lie in the consideration of confinement effect and discretization of the inner steel tube as longitudinal bars under the rule of equal-area. To verify validity and accuracy of the analytical method, the theoretical predictions by the refined method are compared with the experimental results in terms of hysteresis loop, residual drift ratio, and the axial strain of steels.

Chapter Six summarizes the conclusions obtained through chapter two to chapter five, and presents several suggestions to deal with the problems remained to be solved.

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CHAPTER TWO Axial Behavior of Steel-Concrete Composite Short Columns Confined by Circular Thin Steel Tube

2.1 Introduction

Kuratomi et al [2.1] and Kawano et al. [2.2] have proposed a new type of square SC column, consisting of cross-shaped steel or H-shaped steel and concrete confined by thin square steel tubes. In this new type of SC columns, the outer thin square steel tube was only used to confine concrete with the aim of further enhancement of ductility of the columns. Ehab et.al [2.3] investigated the behavior of pin-ended axially loaded concrete encased H-shaped steel composite columns, and developed a nonlinear 3-D finite element model to analyze the inelastic behavior of steel, concrete, reinforcement bars as well as the effect of concrete confinement of the concrete encased H-shaped steel composite columns. The test results reflected that the increase in structural steel strength has a small effect on the composite column strength for the columns having higher relative slenderness ratios due to the flexural buckling failure.

There are still many studies related to SC columns, but most of them placed emphasis on the square SC columns with I-shaped or cruciform steels encased. Furthermore, considering the significantly higher buckling resistance of square steel tubes compared to I-shaped and/or cruciform steels, this paper proposes encasing square steel tube into circular concrete columns confined by thin circular steel tubes to make full use of high confinement effect by circular steel tubes. To promote application of the proposed SC columns, basic mechanical properties of them need to be addressed. To this end, in chapter two, two series of short columns were researched, encompassing not only thin steel tubes fabricated by welding but also a novel type of bolted thin steel tube. The new type of thin steel tube was fabricated by at first bending a flat steel plate with targeted thickness into semi-circular plate with connection wing at both ends of the semi-circular plate, and then clamping two plates by high-strength bolts and nuts along the height of the circular tube.

The primary objectives of this chapter are, 1) to compare the effect of infilling concrete in the inner steel tube on the axial behaviors of the SC short columns and the impact of local buckling

of the inner steel tube, 2) to compare the confinement effect of outer circular thin steel tubes with different joint methods (welded and bolted) on concrete strength, 3) to study the influence of the outer-diameter-to-thickness ratio (111.4 and 78) of circular thin steel tube on the strength enhancement ratio K of confined concrete, and 4) to find a model evaluating the stress-strain relationships of concrete confined by welded and/or bolted circular thin steel tubes.

2.2 Test Programs

2.2.1 Outlines of Test Columns

To clarify the concrete confinement effects by circular thin steel tube in the proposed SC short columns, totally fourteen circular cross-section specimens were fabricated and tested under the monotonic axial compressive tests. The details and outlines of all test short columns are shown in **Fig. 2-1**, **Fig.2-2**, **Table 2-1** and **Table 2-2**. The test short columns were divided into two series. Series 1 includes SC short columns with an inner steel tube encased, and series 2 contains SC short columns with different thicknesses of outer circular thin steel tubes.





Table 2-1 Primary experimental parameters of the series 1 specimens and main experimental results

No.	Section Configuration	Unconfined concrete	Outer circular thin steel tube	Inner square steel tube	Experimental Maximum N (kN)	Calculated Maximum N1(kN)	Experimental K_1	Calculated K
W-C-1					1387	1520	1.21	1.37
B-C-1			\bigcirc		1276	1525	1.08	1.37
W-C-2			Φ175x1.6 mm		1241	1285	1.30	1.37
B-C-2		<i>f</i> _p =36.0	<i>f_{yh}=</i> 185.6MPa	75x75x3.2mm	1180	1278	1.21	1.37
W-C	$\bigcirc \blacksquare$	MPa	(the same for	<i>f_{yt}</i> =436.0MPa	1132	1183	1.31	1.37
B-C	$\cup \cup$		both welded		956	1194	1.10	1.37
P-C-a			and bolted		854	866		
P-C-b			steel tubes)		877	866		_

Note: The calculated K was based on the proposed formula in reference [2.4]. The experimental K1 was obtained by dividing the confined concrete strength by the unconfined concrete strength f_p (the average value of P-C-a and P-C-b). (When calculating the strength of confined concrete, the portion of the inner steel tube was subtracted, and all the inner steel tubes yielded before the peak load, $N_c = N - N_{tube}$, N_c =The axial force of the concrete part of the specimens W-C-1, W-C-2, B-C-1, B-C-2. N_{tube} = the axial force of the inner steel tube)

All specimens of series 1 as shown in **Fig. 2-1** had a height of 350mm and an inner diameter of 175mm. The thickness of welded/bolted outer circular thin steel tubes was 1.6mm in thickness (the outer diameter-to-thickness ratio=111.4). Four specimens W-C-1, B-C-1, W-C-2, and B-C-2 had FA rank square encased steel tubes (width-to-thickness ratio=23.4) and confined by welded and/or bolted outer circular thin steel tubes. Concrete was poured into the square encased steel tube in specimens W-C-1 and B-C-1, while W-C-2 and B-C-2 were left without concrete infilling. Additionally, two concrete short columns W-C and B-C, without square inner steel tube, only confined by the same outer circular steel tube, were tested. To more accurately assess the concrete compressive strength of this series of specimens, two plain concrete short columns P-C-a and P-C-b were cast. These eight specimens were tested to investigate effects of the fabrication method of outer circular steel tube, the presence of inner steel tube, and the infilling of concrete on the axial performance of the proposed SC columns.

No.	Joint method	Thickness of steel tube	Section configuration	Unconfined concrete	Outer circular thin steel tube	Experimenta 1 Maximum N (kN)	Calculated Maximum N1(kN)	N/N1	Experimental K_1	Calculated K
W-C-t1	Welded	1.6mm			\bigcirc	1505	1414	1.06	1.57	1.47
B-C-t1	Bolted	1.6mm	\bigcup		$\Phi_{175x1.6 \text{ mm}}$	1213	1414	0.86	1.26	1.47
W-C-t2	Welded	2.3mm	=()=	f _p =40.0	f _{yh1} =247.3	1578	1520	1.04	1.65	1.58
B-C-t2	Bolted	2.3mm)	MPa	MPa Φ175x2.3mm	1531	1520	1.01	1.59	1.58
P-C-c					f _{yh2} =211.7	965				
P-C-d		_			MPa	951	_		_	_

Table 2-2 Primary experimental parameters of the series 2 specimens and main experimental results

Note: f_p : Average strength of P-C-c and P-C-d. The calculated K was based on the proposed formula in reference [2.4]. The experimental K1 was obtained by dividing the confined concrete strength by the unconfined concrete strength f_p (When calculating the strength of concrete, the portion of the inner steel tube was subtracted, and all the inner steel tubes yielded before the peak load. $N_c = N - N_{tube}$, N_c =The axial force of the concrete part of the specimens W-C-1, W-C-2, B-C-1, B-C-2. N_{tube} = the axial force of the inner steel tube)



Fig. 2- 2 Details of series 2 specimens

Furthermore, to isolate the influence of the inner square steel tubes and focus solely on the confinement effect by the outer circular thin steel tubes with different connection methods and thicknesses, four test specimens W-C-t1, W-C-t2, B-C-t1 and B-C-t2 that comprised series 2 specimens and had no inner square steel tubes encased, were tested under axial compression (As shown in **Fig.2-2** and **Table2-2**). The specimens W-C-t1 and W-C-t2 were confined by welded circular thin steel tube, while the specimens B-C-t1 and B-C-t2 were confined by bolted circular thin steel tube. The t1 and t2 in the notations of these specimens represent the thickness of 1.6mm 2.3mm, respectively. The two thicknesses gave an outer diameter-to-thickness ratios of 111.4 and 78.1, respectively. Likewise, two plain concrete short columns P-C-c and P-C-d were made and tested to reflect the compressive strength of unconfined concrete.

2.2.2 Mechanical Properties of Materials

Based on the test results of the test pieces of used steels, mechanical properties with the tensile stress-strain curves were summarized in **Table 2-3**, **Table 2-4**, separately. For those steels that did not exhibit apparent yield plateau in their stress-strain relations, the yielding strengths of them were determined by the 0.2% offset yielding method.

0.00099

0.00100

one								
PL1.6	Es (GPa)	f_y (MPa)	$f_u(MPa)$	f_y/f_u	Elongation	ε _y		
No.1	186.3	185.4	328.1	0.565	0.442	0.00100		
No.2	180.4	186.1	323.1	0.576	0.440	0.00103		

326.1

325.8

0.568

0.570

0.454

0.445

No.3

AVE

188.1

184.9

185.3

185.6

Table 2-3 Mechanical Properties of the Steels of Series

PL3.2	Es (GPa)	$f_y(MPa)$	$f_u(MPa)$	f_y/f_u	Elongation	ε _y
No.1	196.3	430.2	475.9	0.904	0.270	0.00219
No.2	200.2	440.4	493.1	0.893	0.280	0.00220
No.3	201.1	437.5	488.1	0.896	0.292	0.00218
AVE	199.2	436.0	485. 7	0.898	0.281	0.00219



PL1.6	Es (GPa)	f_y (MPa)	f_u (MPa)	f_y/f_u	Elongation	ε _y	
No.1	193.7	246.3	363.7	0.677	0.418	0.00127	
No.2	194.4	246.1	356.4	0.690	0.426	0.00127	
No.3	194.3	249.4	365.3	0.683	0.407	0.00128	
AVE	194.1	247.3	361.8	0.683	0.417	0.00127	

Table 2- 4 Mechanical Properties of the Steels of Series two

PL2.3	Es (GPa)	fy (MPa)	f_u (MPa)	f_y/f_u	Elongation	ε _y
No.1	184.5	208.1	331.2	0.628	0.496	0.00113
No.2	185.7	214.4	330.7	0.648	0.506	0.00116
No.3	185.0	212.6	327.6	0.649	0.513	0.00115
AVE	185.1	211.7	329.9	0.642	0.505	0.00114



Note Es: Young's modulus, f_u : ultimate stress, f_y : yield stress, ε_y : yield strain= f_y/Es

The specimens were constructed using ready-mixed concrete composed of Portland cement and coarse aggregates with a maximum particle size of 20mm. Based on the test results from three cylinders (with a diameter of 100mm and a height of 200mm) conducted 28 days after casting, the average values of compression strength, Young's modulus, and peak strain for the first batch of eight SC columns (series 1) were 35.1MPa, 30.3 GPa, 0.2%, respectively. Similarly, for the second batch of six short concrete columns (series 2), the average values of concrete compression strength, Young's modulus, and peak strain were 45.3 MPa, 30.8 GPa, and 0.2%, respectively.

2.2.3 Loading Apparatus and Measurements

A monotonic axial compressive load was applied to the specimens via a universal testing machine with a maximum capacity of 2000kN, as shown in **Fig. 2-3.** Two 40mm-thick circular load blocks with a diameter of 170mm were placed at top and bottom of the test specimens to prevent the outer circular thin steel tube from being subjected to the axial force. Additionally, a steel plate was positioned on the top of the upper load block to facilitate the uniform application of axial compression into specimens and the installation of displacement

transducers (DTs). The average axial deformation was measured by four DTs installed between the top steel plate and platform of machine to measure the average axial deformation.

For the specimens shown in **Fig 2-1**, the biaxial strain gauges were embedded on the surfaces of both inner square encased steel tube and outer circular thin steel tube. The strain gauges located at the sections 75mm and 175mm away from the top of the column on both the inner and outer steel tubes. For the inner square steel tube, strain gauges were embedded on the center of its four faces, and for the outer circular thin steel tube, the angle formed between three strain gauges was 120 degrees.

For the series 2 specimens in **Fig. 2-2**, all strain gauges were affixed to the surfaces of the outer circular thin steel tubes. They were positioned at different locations: 15mm, 55mm, 95mm, 135mm, 175mm, and 215mm away from the top of the column. At the same height, two biaxial strain gauges were attached on opposite sides of the diameter of each of the outer circular thin steel tubes. In the case of the bolted outer circular thin steel tube, additional uniaxial strain gauges were attached at the same heights as the biaxial strain gauges in the vicinity of the bending axis of the bolted wing, with the aim of quantifying its deformation.



Fig. 2-3 Loading apparatus

2.3 Experiments Results and Discussion

Steel Tubes



2.3.1 The Damage and Local Buckling of Inner Square Steel Tubes and Outer Circular

W-C-1

W-C-2









Fig. 2-4 shows ultimate failure modes of all square encased steel tubes (surfaces in four directions) of the series 1 specimens after removing the circular thin steel tubes and concrete. Along with the increasing of average axial strains, the major deformation and local buckling of the test specimens W-C-1, W-C-2 and B-C-2 concentrated within the middle and lower regions of the inner square steel tubes. For the specimen B-C-1, in addition to the middle and lower regions, local buckling also occurred in the upper region of the inner steel tube. For the test

specimens W-C-1 and B-C-1 in which concrete was infilled into the inner square steel tube, the inner square steel tubes locally buckled outwards due to existence of concrete, whereas for the specimens W-C-2 and B-C-2 in which no concrete was infilled into the encase steel tubes, the local buckling of the inner square steel tube was inward.

Analyzing the causes of local buckling, the cover concrete spalled off and crushed gradually, which could intensify the local buckling of inner encased square steel tubes. For the hollow inner steel tubes without infilling concrete, local buckling occurred inward under axial compression and the expansion of the covered concrete. For the inner steel tubes infilled with concrete into them, the square inner steel tube provided strong lateral confinement to the infilling concrete, preventing it from serious compression damage. From the situation observed after removing the outer steel tubes, it could be seen that severe compressive damage and collapse was happened of the covered concrete. As a result of the interaction between the infilling concrete and the covered concrete surrounding the inner steel tubes, multiple instances of local buckling occurred outward from the inner steel tubes.

Typically, due to the influence of gravity after concrete pouring, aggregates of concrete tend to subside downward, resulting in higher density and strength in the lower portion of the test specimen compared to the upper portion. This phenomenon increases the likelihood of local buckling occurring in the upper half of the specimens.



Fig. 2-5 Rupture of the circular steel tube of the series two specimens

Fig.2-5 displays the ultimate fracture states of the outer circular thin steel tubes for the series 2 specimens W-C-t1, W-C-t2, B-C-t1 and B-C-t2, none of which was encased with an inner square steel tube. The premature rupture occurred along the welding seams of specimens W-C-t1 and W-C-t2 at a small deformation level. The fracture was also observed along the bending line of the wing of the bolted semi-circular steel tubes in specimens B-C-t1 and B-C-t2. However, unlike in the specimens W-C-t1 and W-C-t2, the fracture along the wing bending line of the bolted circular thin steel tube began at very larger deformation than the welded thin steel tubes.

2.3.2 Axial Load-strain Relationships

The axial load versus average axial strain relationships of series 1 specimens are showed in **Fig. 2-6**, where the average axial strain was obtained by dividing the axial displacement measured by four DTs with the total height of specimens. In the **Fig. 2-6** (a), the black circular point marks represent the fracture moment of the outer welded circular thin steel tubes. In the **Fig. 2-6** (b), the bolted circular steel tubes did not rupture, and the loading was terminated due to the limitation of the displacement meter.



Fig. 2-6 The axial load - average axial strain relationships of series 1 specimens

It can be clearly seen from the **Fig. 2-6** that the confinement by the welded steel tube brought a little higher load-carrying capacity to the SC short columns than the bolted steel tube because the bolted steel tube was not continuous and hence could not provide completely the same

confining pressure to concrete as the welded steel tube. However, after the peak axial load, the load-carrying capacity of the specimens confined by the welded thin steel tubes rapidly decreased, and the welding seams of them suddenly ruptured before the average axial strain of 0.02. On the other hand, as shown in **Fig. 2-6 (b)**, the bolted steel tubes did not experience sudden failure until the axial strain of 0.06, ensuring very stable resistance to the confined concrete. This observation implies that the bolted steel tube can be used to replace the welded steel tube and provide effective and simple confinement for the SC columns.



Fig. 2-7 The average axial strain-axial load relationships of series two specimens

Fig. 2-7 shows the axial load versus average axial strain relationships of series 2 specimens. The black dots superimposed in the **Fig. 2-7** represent the moment when the outer circular thin steel tubes ruptured. As shown in **Fig. 2-7** (a), the specimens W-C-t1 and W-C-t2 reached their peak axial load at the average axial strain of 0.006. After that strain, the welded seams of outer circular steel tube suddenly ruptured before the axial strain reached 0.02, the load-carrying capacity decreased rapidly. Compared the experimental curves of the two specimens with different thicknesses of welded circular thin steel tubes, the ultimate strength of W-C-t2 with a thickness of 2.3 mm (diameter-to-thickness ratio=78) was about 7% higher than that of the specimen W-C-t1 with a thickness of 1.6 mm (diameter-to-thickness ratio=111).
As shown in **Fig. 2-7 (b)**, the specimens B-C-t1 and B-C-t2 reached their maximum axial load near the average axial strain of 0.006. For the specimen B-C-t1, after reaching the peak point, the load-bearing capacity of it suddenly decreased significantly until the average axial strain reached approximately 0.015, after that strain there was a slight increase in the load-bearing capacity. When the average strain was around 0.057, the bending line near the wing of the semicircular bolted steel tube broke, and the loading was terminated. For the test specimen B-C-t2, the load-bearing capacity also significantly decreased after the peak point, but it remained relatively constant after the average axial strain reached 0.03. At the axial strain of 0.07, rupture occurred at the wing of the bolted circular steel tube, marking the end of the loading. The residual load-carrying capacity at large strain for the two specimens confined by the bolted thin steel tube was stable and maintained at around 70% of their peak axial load until the end of loading.

It can be obviously seen from the **Fig. 2-7 (b)** that the maximum load-bearing capacity of B-Ct2 was about 20% higher than that of specimen B-C-t2, which mean that the thicker the bolted circular steel tube, the higher the confinement effect can be expected. Furthermore, as shown in Table 2-2, the confinement efficiency of the bolted steel tube was nearly equal to that of the welded steel tube with the thickness of 2.3 mm, while the strength gain (26%) of concrete confined by thinner bolted steel tube was much less than that (57%) of the welded steel tube. This discrepancy can be attributed to that the alignment of the connection wing at the ends of semi-circular shaped plates was not as precise as the thicker plate.

2.3.3 Strains measured in steel tubes

The measured axial strains of inner square steel tubes and outer circular thin steel tubes for all tested short columns are shown in **Fig.2.8** ~ **Fig.2.3-16**. The horizontal axis R represented the average axial strain which was obtained by dividing the axial displacement measured by four DTs with the total height of specimens. And the title of each graph, for example B-C-1 I-M-1, represents that the strain measured by the strain gauges was located at the middle height section of the inner square steel tube of the specimen B-C-1, which is related to the details provided in

Fig.2.8 (I=Inner, O=Outer, M=Middle, H=High). In each graph, the red solid line represents the result of vertical strain values measured by the biaxial strain gauge, the black dotted line represents the result of lateral strain values measured by the biaxial strain gauge, and the yellow dotted line represents the yield strain value of the steel. Additionally, it should be noted that this chapter involved the calculation of the strength of confined concrete. For the specimens with inner steel tubes, the total axial load-carrying capacity was subtracted from the loading carrying capacity of the inner steel tube. The reason for this calculation is that all strain curves in section 2.3.2 were compared with the load-displacement curves of the corresponding specimens, and it was found that all inner steel tubes entered the yield stage before the peak axial load of the test specimens.

As shown in the **Fig.2.8** ~ **Fig.2.12**, for all series 1 test specimens, there was a significant difference in the strain versus average axial strain relationship of the test specimen between the inner square steel tube and outer circular thin steel tube. Taking the specimen B-C-1 as an example, the inner square steel tube yielded at an average axial strain of 0.006, corresponding to the moment when B-C-1 reached its peak axial load. Afterwards, until the average axial strain reached 0.01, the strain of the inner steel tube showed a sudden decrease, and continued to increase with subsequent loading (as shown in B-C-1 I-M-1, B-C-1 I-M-2, B-C-1 I-M-3, B-C-1 I-M-2, etc.). This phenomenon occurred due to local buckling of the inner steel tube, which caused a sudden change in the strain of the steel tube. By comparing the load average strain curve, it can be seen that the occurrence of local buckling occurred after the peak axial load, indicating that the local buckling of the steel tubes did not affect the maximum axial load-carrying capacity of the specimen.





Fig. 2-8 Strain curves of inner and outer steel tubes of specimen B-C-1

Continuing to observe the strain distribution of the outer steel tube using B-C-1 as an example, and comparing the axial load-average axial strain experimental curve of the specimen B-C-1, it can be seen that the outer steel tube yielded after the peak axial load of the specimen, and the transverse strain values of the outer steel tube was greater than the longitudinal strain values, indicating that the outer steel tube of the specimen fully exerted the transverse confinement effect on the concrete after the peak axial load until the end of loading.





Fig. 2-9 Strain curves of inner and outer steel tubes of specimen B-C-2





Fig. 2- 10 Strain curves of outer steel tubes of specimen B-C





Fig. 2- 11 Strain curves of inner and outer steel tubes of specimen W-C-1





Fig. 2- 12 Strain curves of inner and outer steel tubes of specimen W-C-2

As shown in the **Fig 2.13** \sim **Fig 2.16**, for all series two test specimens, the title of each graph, for example B-C-t1 A1, represents that the strain measured by the strain gauges was located at the top position of the outer steel tube, which was related to the details provided in **Fig 2.2**. In each graph, the red solid line represents the result of vertical strain values measured by the biaxial strain gauge, the black dotted lines represents the result of lateral strain values measured by the biaxial strain gauge, and the yellow solid line represents the yield strain value of the steel.





Fig. 2-13 Strain curves of outer circular steel tubes of specimen B-C-t1

Continuing to use the specimen B-C-T1 as an example, it can be seen that the two uppermost positions on the web side of the bolted circular steel tube had significantly yielded. When the axial strain was 0.038, there was a sudden change in the strain value at position A1. Based on the failure status of the bolted steel tube after the end of loading, it could be speculated that the local bulging occurred at the A1 position of the bolted steel tube. The strain values at other positions of the bolted steel tube of the B-C-T1 had just reached the yield strain value, indicating that the confinement effect of the bolted steel tube had not been fully exerted.





Fig. 2-14 Strain curves of outer circular steel tubes of specimen B-C-t2





Fig. 2-15 Strain curves of outer circular steel tubes of specimen W-C-t1





Fig. 2- 16 Strain curves of outer circular steel tubes of specimen W-C-t2

2.3.4 Confinement effect of circular thin steel tubes

Sun et al. [2.4-2.5] conducted axial compression tests on high-strength concrete short columns confined by circular and square steel tubes with varying thicknesses, and proposed the stress-strain curve models for confined concrete based on the experimental results as follow:

$$Y = \frac{AX + (D-1)X^2}{1 + (A-2)X + DX^2}$$
(2-1)

Where, $X = \varepsilon_c / \varepsilon_{c0}$, $Y = f_c / f'_{cc}$, f_c and ε_c are the stress and strain, f'_{cc} and ε_{c0} are the stress and strain at the peak, $A = E_c / E_{sec}$, $E_c = (0.69 + 0.332 \sqrt{f'_c}) \times 10^4$ is the Young's modulus of elasticity of concrete, $E_{sec}=f'_{cc}/\varepsilon_{c0}$ is the secant modulus at the point of peak stress, and D is the parameter governing the slope of descending portion of the stress-strain curve. The proposed stress-strain curve model can be applied to predict the flexural behavior of confined concrete columns under axial load and bending moment with moment gradient (shear), and a semiempirical formula to estimate the compressive strength of concrete confined by welded circular steel tubes was proposed as Eq. (2-2).

$$f_{cc}' = f_p + 4.1f_r = f_p + 4.1 \cdot \frac{2t}{D - 2t} f_{yt}, \quad K = \frac{f_{cc}'}{f_p}$$
(2-2)

Where f_{cc} ' is the strength of confined concrete, D, t, and f_{yt} are the outer diameter, the thickness, and the yield strength of the outer steel tube, respectively. The parameter K defined in Eq. (2-2) is an index to measure confinement degree by transverse reinforcement, and generally referred to as strength enhancement ratio. The experimental strength enhancement ratio KIpresents the ratio of confined concrete strength f_{cc} ' (Removing the axial load of inner steel tubes which were yielded, $N_c = N_0 - N_{tube}$) to unconfined concrete strength f_p . The experimental results of confined concrete strength can be obtained by dividing the axial load sustained by concrete by the concrete area and are listed in **Table 2-5** and **Table 2-6** for S1 and S2 specimens, respectively.

, , , , , , , , , , , , , , , , , , ,							
No.	Experimental K1	Calculated K	Strength gain Δf_{p1} (MPa)	$\begin{array}{c} \text{Strength gain} \\ \Delta f_p(\text{MPa}) \end{array}$	$\Delta f_{pl}/\Delta f_p$		
W-C-1	1.21	1.37	7.56	13.32	0.57		
B-C-1	1.08	1.37	2.88		0.22		
W-C-2	1.30	1.37	10.8		0.81		
B-C-2	1.21	1.37	7.56		0.57		
W-C	1.31	1.37	11.16		0.84		
B-C	1.10	1.37	3.6		0.27		
Note: Δf_{p1} = The experimental strength gain; Δf_p = the calculated strength gain by Eq. (2-2)							

Table 2-5 Confinement efficiency in series 1 specimens

No.	Experimental	Calculated	Strength gain	Strength gain	$\Delta f_{pl}/\Delta f_p$	
	K1	Κ	Δf_{p1} (MPa)	$\Delta f_p(MPa)$		
W-C-t1	1.57	1.47	22.8	18.8	1.21	
B-C-t1	1.26	1.47	10.4		0.55	
W-C-t2	1.65	1.58	26.0	23.2	1.12	
B-C-t2	1.59	1.58	23.6		1.02	

Table 2- 6 Confinement efficiency in series 2 specimens

From **Table 2-5**, it is obvious that the strength gains of concrete confined by welded steel tubes are higher than those of concrete confined by bolted steel tubes in series 1 specimens. The strength gains of the specimens confined by bolted tubes were 38%, 70%, and 32%, having an average of 46.7% of the specimens confined by welded tubes. However, even the specimens W-C-1, W-C-2, and W-C confined by welded tubes didn't reach the calculated strength of confined concrete by Eq. (2-2). One of the main reasons for this discrepancy can be attributed to that the thin steel plate used to fabricate the steel tubes, both welded and bolted, didn't reach the specific yield strength of SS 400 steel.

It is noteworthy from **Table 2-6** that confined concrete strength in specimens W-C-t1 and W-C-t2 was 12%-21% higher than the calculate strength by Eq. (2-2). This observation implies that if the thin steel plates, which were used to make the welded steel tubes, are normal JIS product (see **Table 2-1** and **Table 2-2**), the strength gain or confinement efficiency of the welded thin circular steel tubes could be satisfactorily evaluated by Eq. (2-2). In the specimens confined by bolted steel tubes, the strength gains of confined concrete were 55% and 102% of the calculated strength by Eq. (2-2), and 46% and 91% of the counterpart specimens confined by welded tubes, when the wall thickness of the steel plate was 1.6mm and 2.3mm, respectively. The value of 46% obtained in series 2 specimen B-C-t1 is very close to the average strength gain ratio of 46.7% observed in series 1 specimens confined by bolted thin steel tube with the same thickness (1.6mm) as that used in specimen B-C-t1.

On the other hand, when confined by thicker bolted steel tube with thickness of 2.3mm, the strength gain in concrete confined by bolted steel tube was very close to that of concrete

confined by welded steel tube with identical wall thickness and beyond the calculated strength gain by Eq. (2-2). The large discrepancy in the confinement efficiency provided by bolted steel tubes can be attributed to the fabrication precision of them. As observed and measured during preparation of test columns, the alignment of the connection wings at both ends of semi-circular plates with thickness of 1.6mm was not as precise as that of semi-circular plates with thickness of 2.3mm, which may disturb the uniform distribution of lateral confining pressure by the thinner bolted tubes and reduce confinement efficiency.

Based on the above-mentioned discussions on test and theoretical results, it is reasonable to presume that the thicker bolted circular steel tubes with D/t ratio smaller than 78 could provide nearly the same confinement effect to concrete as the welded circular tubes, but the confinement efficiency (strength gain of confined concrete) of the thinner bolted steel tube with D/t ratio larger than 111 should be taken as 50% of that provided by the welded steel tube with identical D/t ratio.

2.4 Comparison of stress-strain curves of concrete confined by circular steel tubes

Fig. 2-17 compares the experimentally measured stress-strain curves with the calculated ones for series 1 specimens, while comparisons between the experimental and calculated stress-strain curves are illustrated in **Fig. 2-18** for series 2 specimens. The theoretical stress-strain curves depicted in these two figures are obtained by using Eq. (2-1). Based on the observation made in previous section, the compressive strength of concrete confined by welded circular steel tubes will be evaluated by using Eq. (2-2). For the concrete confined by bolted steel tubes, its compressive strength will be calculated by Eq. (2-2) when the D/t ratio of the steel tube is smaller than 78, but the strength gain is only taken as half of that calculated by Eq. (2-2) when the D/t ratio of the steel tube is larger than 111. In other words, the strength enhancement ratio K1 for the concrete confined by bolted thinner steel tube will be evaluated as follow:

$$K1 = 1 + 0.5(K - 1), \quad K = \frac{f'_{cc}}{f_p}$$
 (2-3)



Fig. 2- 17 Comparisons of experimental and calculated stress-strain curves of series 1 specimens As obvious from **Fig. 2-17**, for series 1 specimens, the calculated stress-strain curve showed satisfactorily good agreement with the experimental result for the concrete confined by welded steel tube that did not rupture until axial strain of 0.04. For the concrete confined by welded steel tube that prematurely ruptured at small axial strain, the descending portion in the calculated stress-strain curves did not trace the experimental results. However, for the concrete confined by bolted thin steel tube that did not prematurely rupture, taking confinement

efficiency by the bolted thin steel tube as half of that by welded tube could make the theoretical stress-strain curves trace the experimental curves fairly well up to large strain.

For series 2 specimens, the calculated stress-strain curves of concrete confined by welded steel tubes exhibited very good agreement with the experimental curves until the strain where the welding seams began rupturing. On the other hand, for concrete confined by bolted thin steel tubes, the calculated stress-strain curves predicted the experimental results fairly well up to as large strain as 0.06. Abrupt decrease in the axial stress was observed in the specimens B-C-t1 and B-C-t2, which were confined by bolted steel tubes. This phenomenon can be attributed to the discontinuity of the bolted steel tube along the perimeter of cross section, which tends to delay confinement action of the outer steel tube soon after the infilled concrete begins to dilate bear its peak resistance (strength). However, by comparing the whole stress-strain responses one can see that this disadvantage does not hinder application of the bolted tubes to practice.



Fig. 2- 18 Comparisons of experimental and calculated stress-strain curves of series 2 specimens

2.5 Conclusions

A new type of SC column was proposed in this paper. The proposed SC columns consists of encased square steel tube and outer circular steel tubes, welded and bolted. In order to obtain fundamental information on the mechanical property of concrete confined by welded and bolted circular thin steel tubes, a total of ten short concrete columns were made in two batches and tested under concentric loading. From the experimental results described in this chapter, the following conclusions can be drawn:

- (1) Encasing FA rank square steel tube with concrete infilled could enhance both strength and ductility of concrete and well utilize the confinement effect by the outer circular thin steel tubes, both welded and bolted. Without concrete infilled, local buckling of the encased square steel tube tended to decrease ductility of concrete.
- (2) Confinement by the bolted circular steel tubes were superior to that by the welded thin steel tubes from the perspective of enhancement of ductility of concrete. The bolted thin steel tubes did not rupture until as large strain as about 0.06, ensuring secure confinement effect to concrete. On the other hand, the welded thin steel tubes tended to prematurely rupture along the welding seams, implying that careful attention must be paid to the welding process.
- (3) Confinement efficiency of the welded circular thin steel tubes could be accurately evaluated by the formula (Eq. (2-2)) proposed in previous study if the steel plates meet requirement of the JIS. Confinement effect of the bolted thin steel tubes could also be evaluated by Eq. (2-2) when the D/t ratio was smaller than 78, but the confinement efficiency of the bolted tubes with D/t ratio larger than 111 should be assumed as half of that by welded tubes for conservativeness.
- (4) The stress-strain relationships of concrete in the proposed SC columns could be evaluated by Eq. (2-1) through Eq. (2-3) with very good accuracy. In particular, for the SC columns confined by the bolted steel tubes, the calculated stress-strain curves traced the experimental

curves very well until axial strain of about 0.06.

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CHAPTER THREE Structural Performance of Square Steel Tube Encased Concrete Columns Confined by Welded Circular Thin Steel Tube

3.1 Introduction

Based on the axial compression tests conducted on SC short columns in Chapter two, it could be seen that encasing FA rank square steel tube with concrete infilled could enhance both strength and ductility of concrete and well utilize the confinement effect by the outer circular thin steel tubes. However, as the inner steel tubes approach to FA rank, the cost performance becomes worse. In addition, when the square inner steel tube was not filled with concrete, local buckling of the encased square steel tube was easily to occur and influence the ductility of specimens. Therefore, to 0make more economical and reliable design of the proposed SC columns, it is necessary to clarify the influence of grades of the inner square steel tubes on the seismic performance of SC columns, and obtain basic data on the impact of the infilling concrete into inner square steel tube.

Objectives of this chapter are: 1) to obtain fundamental experimental information on structural performance of the square steel tube encased concrete columns confined by welded circular thin steel tubes, 2) to investigate the effects of the grades of encased square steel tubes (FA rank, FB rank, and FC rank) on structural performance of the proposed SC columns, 3) to study the influence of the infilling concrete within the inner square steel tubes, and 4) to verify the confinement effect on concrete by the welded circular thin steel tubes with an outer-diameter-to-thickness ratio of 189.

3.2 Experimental program

3.2.1 Outlines of test specimens

To obtain fundamental data on structural performance of the proposed circular SC columns confined by welded thin steel tubes with outer diameter-to-thickness ratio of about 189, thirteen specimens were fabricated and tested under cyclically reversed lateral force and constant axial compression. The experimental variables included the axial load ratio (0, 0.15, 0.25), the grades of inner square steel tubes (FA rank, FB rank and FC rank), the infilling concrete into the inner steel tube or not, and the presence or absence of main longitudinal rebars.

The details and primary test results are shown in **Fig.3-1** and **Table 3-1**. All specimens were 1/3 scale circular columns with inner diameter of 300mm, and the shear span of the specimens was 910mm to give a shear span ratio of about 3.0. The thickness of outer thin steel tubes of all specimens was 1.6mm (outer diameter to thickness ratio=189.5). The encased inner steel tubes of FA rank, FB rank and FC rank represent sound ductile, moderate ductile and low ductile steel tubes, respectively.



Fig. 3-1 Details of Specimens



(b) Series B/C (base of column was stub)

Fig. 3-1 Continued

For the specimens of series A: As shown in **Fig.3-1** (a), concrete was infilled into the inner square steel tube for A-15F, A-00F, A-25F, A-15FR, and was not infilled in specimen A-15VR. The inner steel tube was fixed to the baseplate by welding, and a clearance of 9 mm was provided between the outer steel tube and the baseplate to ensure that the outer steel tube only provide lateral confinement. The square inner steel tube of FA rank had a side length of 120mm and a thickness of 6mm. Four D6 rebars were placed only in specimens A-15FR and A-15VR. The outer thin steel tube for confining concrete was fabricated by at first bending a flat steel plate with target thickness into circular shape and then welding the seam by laser. The diameter (D) to the thickness (t) ratio is as small as 189.5.

For the specimens of series B and C: As shown in **Fig.3-1(b)**, concrete was infilled into the inner square steel tube for B-15F, C-15F and C-25F, and was not infilled in specimen B-15V, B-25V, B-15VR, C-15V and C-25V. The encased inner steel tubes of FB rank and FC rank

	Notation	Section Configuration	Inner Tube	Infilling Concrete	Outer Tube	Rebars	$\begin{array}{c} f_c' \\ (\text{MPa}) \end{array}$	п
	A-15F		FA Rank	0	Ф303.2x1.6	_	39.2	0.15
sA	A-00F			0			38.7	0.00
Serie	A-25F		120x120x6 HBL385	0	SPCC $f_{yp}=234.3$		41.9	0.25
	A-15FR		f_{yt} =449.0 MPa	О	MPa	4-D6 (SD295) <i>f_{ys}=369.7</i> MPa	41.9	0.15
	A-15VR			Х			41.3	
	B-15F		FB Rank \Box 150×150×4.5 STKR400 f_{yt} = 383 MPa	0	$\Phi 303.2 \times 1.6$ SPCC $f_{yh} = 237$ MPa	_	40.0	
	B-15V			Х			41.4	
and C	B-15VR			Х		4-D13 (SD295) <i>f_{ys}=330</i> MPa	40.3	0.15
ries I	B-25V			Х			42.0	0.25
Se	C-15F		FC Rank \Box $175 \times 175 \times 4.5$ STKR400 $f_{yt} = 377$ MPa	0			42.1	0.15
	C-15V			Х			43.8	
	C-25F			0			42.5	0.25
	C-25V			X			42.6	0.23

Table 3-1 Details and primary test results of specimens

Note: f_{yt} =yield strength of inner tube; f_{yh} =yield strength of outer tube; f_{ys} =yield strength of rebars; f_c' =concrete compressive strength; N=actual axial force; $N_o = f_c' A_c + f_{yt} A_{st}$; $n = N/N_o$

have square cross-sections with a thickness of 4.5mm and outer widths of 150mm and 175mm, respectively. The height of the loading stub of each specimen was 500 mm, with the inner square steel tube welded to a baseplate (with a thickness of 22mm) placed at the bottom of the stub. A clearance of 9 mm was provided between the outer welded circular steel tube and the upper surface of loading stub to guarantee that the outer welded circular steel tube only provides lateral confinement without bearing constant axial force. The outer welded thin steel tube for confining concrete was fabricated by laser-welding the seam.

The axial load ratio represents the ratio of applied axial force (N) to the nominal axial capacity (N_o) of SC columns section. As shown in the note of **Table3-1**, the nominal axial capacity N_o was calculated by simply summing the axial capacities of the encased (inner) steel tube and the concrete portion.

3.2.2 Material properties

The specimens were fabricated by using ready-mixed concrete and coarse aggregates with a maximum particle size of 20mm. Based on the test results of three cylinders (100mm in diameter and 200mm in height), the average values of series A specimens of 28-days compressive strength, young's modulus and peak strain of the concrete were 39.2 MPa, 31.5 GPa, and about 0.2%, respectively. For series B and C specimens, the 28-days compressive strength, young's modulus and peak strain of the concrete were 41.7 MPa, 33.5 GPa, and 0.2%, respectively.

The mechanical properties of the FA rank encased steel tube and the welded circular thin steel tube are shown in **Table.3-2.** For those that did not exhibit apparent yield plateau in their stress-strain relations, the yielding strengths of them were determined by the 0.2% offset yielding method. The yield strain ε_y is calculated through dividing the yield stress by the Young's modulus. For the FA rank inner steel tube, the yield strain was 0.0022, the yield strain of outer welded steel tube and D6 rebars were 0.0012 and 0.0019, respectively.

The mechanical properties of the FB rank and FC rank encased steel tube and the welded circular thin steel tube are shown in **Table.3-2**. For the FB and FC rank inner steel tubes, the yield strains were 0.0019. The yield strain of the outer welded thin steel tube and D13 rebars were 0.0012 and 0.0017, respectively.

Table 3-2 Mechanical Properties of the Steels of Series A, B and C

The specimens of series A

Inner Steel Tube FA Rank (HBL385)							
PL6	Elongation	E_s (GPa)	$f_y(MPa)$	$f_u(MPa)$	f_y/f_u		
No.1	0.343	200.70	447.25	583.68	0.766		
No.2	0.350	202.80	449.05	584.56	0.768		
No.3	0.372	202.60	450.70	585.77	0.769		
Ave	0.355	202.03	449.00	584.67	0.768		

Outer Thin Steel Tube PL1.6 (SPCC)

200.38

197.06

197.19

198.21

 $f_y(MPa)$

234.76

234.67

234.58

234.67

 $f_u(MPa)$

342.60

346.14

343.68

344.14

 f_y/f_u

0.685

0.678

0.683

0.682

Elongation E_s (GPa)

0.428

0.419

0.398

0.415

PL1.6

No.1 No.2

No.3

Ave



Rebars D6 SD295							
D6	Elongation	E_s (GPa)	$f_y(MPa)$	<i>f_u</i> (MPa)	f_y/f_u		
No.1	0.194	194.43	378.06	529.38	0.714		
No.2	0.144	184.61	361.13	519.43	0.695		
No.3	0.161	194.67	369.18	521.17	0.708		
Ave	0.166	191.24	369.45	523.33	0.706		

The specimens of Series B and C



Inner Steel Tube FB Rank (STKR400)							
PL4.5	Elongation	E_s (GPa)	f_y (MPa)	f_u (MPa)	f_y/f_u		
No.1	0.371	199.85	362.00	440.70	0.821		
No.2	0.294	201.46	402.10	454.23	0.885		
No.3	0.347	204.56	383.21	449.05	0.853		
Ave	0.337	201.96	382.44	447.99	0.853		

Inner Steel Tube FC Rank (STKR400)							
PL4.5	Elongation	E_s (GPa)	f_y (MPa)	f_u (MPa)	f_y/f_u		
No.1	0.322	200.81	375.28	450.15	0.834		
No.2	0.359	202.48	378.76	449.18	0.843		
No.3	0.395	250.52	377.58	451.35	0.837		
Ave	0.359	201.65	377.21	450.23	0.838		

Outer Thin Steel Tube (SPCC)

196.73

198.10

203.45

199.43

 f_y (MPa) f_u (MPa)

346.09

352.23

349.32

349.21

236.90

237.80

236.64

237.11

 f_y/f_u

0.685

0.675

0.677

0.679

PL1.6 Elongation E_s (GPa)

0.482

0.478

0.473

0.478

No.1

No.2

No.3

Ave



Fig.3- 2 Stress-strain curves of steels used

	D-13 Rebars								
D-13	Elongation	E_s (GPa)	f_y (MPa)	f_u (MPa)	f_y/f_u				
No.1	0.224	194.99	328.17	454.86	0.721				
No.2	0.243	194.24	330.89	458.17	0.722				
No.3	0.233	181.56	329.46	457.86	0.720				
Ave	0.233	190.26	329.51	456.96	0.721				

3.2.3 Test setup and loading program



(a) Loading apparatus of series A specimens

(b) Loading apparatus of series B/C specimens

Fig. 3-3 Loading apparatus



Fig. 3- 4 Loading program

As illustrated in the **Fig.3-3**, cyclical reversed horizontal force and constant axial compression were applied to all specimens via hydraulic jacks. A vertical hydraulic jack with a capacity of 1000 kN and connected to a stiff loading frame via a linear slider, was utilized to apply constant axial compression. Simultaneously, a 500kN capacity jack was adopted to apply the cyclically reversed lateral force. The cyclic lateral force was controlled by the drift angle R, which is defined as the ratio of the lateral tip displacement to the shear span. The targeted loading program is shown in **Fig.3-4** Two complete loading cycles were applied at each level of the targeted drifts (0.005 rad, 0.01 rad, 0.015 rad, and 0.02 rad), and one cycle was applied at each level of the targeted drift after drift angle exceeded 0.02 rad.

3.2.4 Instrumentations and measurement



Fig.3- 5 Strain gauge position



Fig.3- 6 DTs position of series A Figure



Fig.3-7 DTs position of series B/C

For each specimen of series A, a total of thirty-two strain gauges were embedded on the surfaces of both inner encased square steel tube and circular outer steel tube as shown in Fig.3-5. The strain gauges located at the sections 30mm, 150mm, 300mm, and 600mm away from the column bottom. For specimens A-15FR and A-15VR, four more strain gauges were attached to the longitudinal D6 rebars at the same height as that of the strain gauges measuring the strains of steel tubes. For the specimens of series B and C, except for the absence of strain gauges at 600mm, the positions of other strain gauges remained consistent with those of series A specimens. As shown in Fig. 3-6, five displacement transducers (DTs) were used to measure the lateral displacement of specimens of series A. The average of the displacement measured by DTs No. 1 and No. 2 was adopted to present the lateral tip displacement for all specimens. The No. 3 DT was used to measure the slip of the baseplate, while DTs No. 4 and No. 5 were adopted to measure the potential rotation of the endplate. As shown in Fig. 3-7, four DTs were used to measure the lateral displacement and the axial shortening or elongation of specimens. The average of the displacements measured by DTs No. 1 and No. 2 were adopted to present the lateral tip displacement for specimens of series B and C, the other two DTs No.3 and No.4 were installed to track the vertical displacement of the specimens, and the average values of their measurements were taken as the axial deformation results.

3.3 Test Results and Observation



3.3.1 Horizontal Force (V)–Drift Angle(R) Relationships

Fig. 3- 8V-R relationship of specimens in series A

For the specimens of series A:

The measured V-R relationships of all specimens in series A are shown in **Fig 3-8**. The red solid lines and blue dotted lines superimposed in **Fig 3-8** represent the mechanism lines based on the ultimate flexural strengths calculated by the design methods recommended in SRC standard [3.1], design guideline for concrete filled steel tubular (CFT) structures of AIJ [3.2], and the modified CFT equation by taking account of confinement effect by the outer circular

thin steel tubes, respectively. The red solid circles express the peak points. As one can see from **Fig 3-8**, all specimens exhibited stable cyclic response up to large drift angle of at least 0.04 rad and reached their ultimate lateral load capacities that took confinement effect into account. The larger the axial compression, the higher the maximum lateral resistance. For all specimens of series A, the encased square steel tubes reached the yield strength at the drift angle of 0.015 rad. The hysteresis loops before that drift angle exhibited inversed S-shape, indicating that concrete played dominant role in resisting the lateral force before R reached 0.015rad. From R=0.015 rad on, the V-R hysteresis loop became rounder along with drift angle, implying larger energy dissipation capacity due to the yielding of encased steel tubes.

(a) Effect of the axial load ratio:

The specimens A-15F, A-15FR, and A-15VR which were under identical axial compression with axial load ratio n=0.15, reached their maximum lateral forces around the drift angle of 0.03 rad. After that drift angle, their lateral resistance declined gradually and in parallel to the mechanism line, showing very ductile behavior up to drift angle of over 0.07 rad. The loading for these specimens was terminated during being loaded towards0.08 rad due to the rupture of the welding seams of the outer circular thin steel tubes. In specimen A-00F, which was not subjected to axial compression, the wielding seam of circular steel tube didn't rupture and the lateral load was terminated at drift angle of about 0.09 rad due to the limit of the loading apparatus. The increase in lateral resistance at large drift can be attributed to the strain-hardening effect of the encased steel tube. The specimen A-25F under high axial compression with axial load ratio of 0.25 reached its maximum lateral force at R=0.03 rad. After that drift angle, the lateral resistance declined almost in parallel to the mechanism line. During the loading towards drift angle of 0.05 rad., the wielding seam of the outer circular steel tube ruptured at R=0.04 rad and the test was terminated. Significant bulge was observed in the circular thin steel tube from R=0.03rad on.

(b) Effect of infilling of concrete into the encased steel tube

From the test results of the specimens A-15FR and A-15VR and the skeleton curves in **Fig 3-8**, the difference in hysteresis curves and skeleton curves between A-15FR and A-15VR was very small, so it is clear that infilling concrete into the encased steel tube could increase the lateral resistance but didn't significantly influence the hysteresis response. This phenomenon can be attributed to that all of the encased tubes in the tests were FA rank ones, which prevented them from local buckling and affecting the hysteresis behavior of the SC columns.

(c) Effect of D-6 rebars

By comparing behaviors of specimens A-15F and A-15FR in **Fig 3-8**, one can see that there was little difference between their hysteresis loops, the shape and areas of their hysteresis curves were very similar, but there was a slight difference in their maximum load-carry capacity, and placement of longitudinal rebar only enhanced the maximum lateral force by about 4%. From these results, it can be seen that the increase in load-carrying capacity caused by setting the main rebars in this kind of specimens is relatively limited.




Fig. 3- 9V-R relationship of specimens in series B and C

For the specimens of series B and C:

The measured V-R relationships of all specimens in series B and C are shown in **Fig 3-9**. The red solid lines, blue dotted lines and green chain lines superimposed in **Fig 3-9** represent the mechanism lines based on the ultimate flexural strengths calculated by the design methods recommended in SRC standard, the design guideline for concrete filled steel tubular (CFT) structures of AIJ, and the modified design guideline for CFT structures (it will be described in detail in chapter five) taking the confinement effect into account respectively.

For the specimens B-15F and C-15F, they reached the yield strength at the drift angle of 0.015 rad, while other six specimens B-15V, B-25V, B-25VR, C-15V, and C-25V reached their yield strength at R=0.01 rad. After that, the *V*-*R* hysteresis loop became rounder with a larger energy dissipation capacity with the yielding of inner steel tubes. Except for the specimen B-15F, the load-carrying capacity of other seven specimens sharply decreased with the rupture of the welding seams of the welded outer steel tube and the loading ended. The load-carrying capacity of the C-15V and C-25V decreased significantly faster than that of the P- Δ mechanism lines.

After loading ended, the outer steel tubes and the covered concrete were peeled off, and the inward local buckling was observed in the inner steel tubes.

(a) Effect of grades of inner steel tubes:

Comparing the specimens in **Fig 3-9**, where all other parameters remained the same and only the grade of the inner steel tube varied (e.g. B-15F and C-15F, B-15V and C-15V, B-25V and C-25V), it becomes evident that the maximum load-carrying capacity of the specimens encased with an FB rank inner steel tube was 89% to 93% of that of the specimens with an FC rank inner steel tube. This difference in maximum load-carrying capacity was due to the difference in the cross-sectional areas of the FB and FC rank inner steel tube when the yield strength of the inner steel tubes and the concrete strength were similar.

Comparing specimens B-15F and C-15F, there was a sudden decrease in load-bearing capacity of C-15F at R-0.05 rad due to the rupture of the outer steel tube, indicating more severe concrete deterioration in the extensive deformation compared to B-15F.

For the specimens B-15V and C-15V without infilling concrete into inner steel tube, B-15V exhibited a fuller hysteresis loop shape with higher energy dissipation capacity until R=0.05 rad, while C-15V showed a significant decline in load-bearing capacity after the peak lateral load and eventually terminated loading around R=0.03 rad due to the rupture of the welded seam of outer steel tube.

For the specimens B-25V and C-25V with an axial load ratio of 0.25 and no concrete infilling into inner steel tube, their hysteresis performance was poor and exhibited significant brittleness. After R=0.02 rad, the loading was terminated due to the sudden fracture of the outer steel tube.

In summary, specimens encased with a FC rank inner steel tube exhibited more pronounced brittleness during the loading process, with the rupture of the outer steel tube being a common occurrence. In the conditions of having infilling concrete and low axial load ratios, the specimens with FB rank and FC rank inner steel tubes all maintained relatively great hysteresis performance and load-carrying capacity up to R=0.05 rad, and the specimen with FB rank inner steel tube showed better ductility compared to the specimen with a FC rank inner steel tube. However, as the axial compression ratio increased and the infilled concrete was removed, the seismic performance of both series B and series C specimens significantly decreased, leading to noticeable concrete and outer thin steel tubes damage at smaller deformations.

(b) Effect of the axial load ratios:

The impact of the axial load ratios could be researched by comparing specimens B-15V and B-25V, C-15F and C-25F, as well as C-15V and C-25V. For the specimens B-15V / B-25V and C-15F / C-25F, the specimens with an axial load ratio of 0.15 were all loaded up to R=0.05 rad. The specimens with an axial load ratio of 0.25, experienced a sudden rupture of the outer steel tube during the loading process of R=+0.03 rad when R reached around 0.02 rad, resulting in the end of the loading and exhibiting obvious brittle characteristics. The larger the axial load ratio, the greater the value of maximum lateral load.

Additionally, specimens C-15V and C-25V experienced outer steel tubes damage and load termination before reaching R=0.03 rad, with the rate of load decline significantly exceeding the P- Δ mechanism lines. This was attributed to the occurrence of local buckling in both of the two specimens. Therefore, for specimens with FC rank hollow inner steel tubes encased, their inherent seismic performance was relatively poor, with the axial load ratio having a less pronounced effect on the shape of their hysteresis curves. However, an increase in the axial load ratio resulted in an approximately 17% increase in load-bearing capacity.

(c) Effect of infilling of concrete into the encased steel tube

This chapter investigated the impact of infilling concrete by comparing specimens B-15F and B-15V, as well as C-15F and C-15V. According to compare the results of B-15F and B-15V, it could be seen that infilling concrete could increase the maximum lateral load by 15%, and the

specimen B-15F had good ductility and was loaded until R=0.1 rad. Meanwhile, For the specimens encased with a FC rank inner steel tube with an axial load ratio of 0.15, infilling concrete could increase the maximum load-carrying capacity by 21%. Furthermore, the specimens encased with a hollow FC rank inner steel tube were prone to occur inward local buckling, whereas the presence of infilling concrete ensured that the specimen C-15F maintain a reasonable load-bearing capacity above the calculated value until R=0.05 rad.

(d) Effect of D-13 rebars

Comparing the V-R relationships of specimens B-15V and B-15VR, it could be seen that the two specimens exhibited similar hysteresis shapes and energy dissipation performance, and four D13 steel bars could increase the maximum lateral load of B-15V by 11%.





Fig. 3- 10 The drift angle-vertical displacement relationship of specimens with FB and FC rank inner steel tube encased

Fig.3-10 illustrates the vertical displacement versus drift angle relationship of the welded specimens with FB rank inner steel tubes and FC rank inner steel tubes encased. As shown in the Fig.3-10, except for the C-15V and C-25V test specimens, the vertical compression deformations at R=0 rad for each hysteresis loop of all other test specimens were within 1mm. The vertical deformation of the C-15V specimen was all in a compressed state, and the maximum vertical displacement corresponding to its target drift angle was suddenly increased from the targeted drift angle of 0.02 rad, which was consistent with the moment when its lateral resistance suddenly decreased in Fig.3-9. The maximum vertical compression deformation reached 2.5 mm after the hysteresis loop with R=0.03 rad. For the specimen C-25V, after the drift angle was -0.02 rad, the vertical compression deformation of C-25V suddenly increased significantly. By the second cycle of R=0.02 rad, the vertical compression deformation had exceeded 4 mm. After all the test specimens were loaded, outer welded thin steel tubes were removed and the cover concrete was peeled off to observe the deformation of the FB rank and

FC rank inner steel tubes. It was found that the specimens C-15V and C-25V exhibited obvious inward local buckling near the bottom of the column. In general, for specimens with the FC rank hollow inner steel tube encased, inward local buckling of FC rank inner steel tube was more likely to occur, while for specimens with concrete infilled into the FC rank inner steel tube, the infilling concrete expansion within the inner steel tube facilitated interaction with the cover concrete, reducing the susceptibility to local buckling.

3.3.2 Residual Drift Angle



Fig. 3-12 Residual drift angle of specimens in series B and C

Fig. 3-11 and Fig. 3-12 show the average residual drift angle of the initially push direction measured at each targeted drift level. One can see form Fig. 3-11 that no difference was

observed in the measured residual deformation among specimens of series A until drift angle reached 0.015 rad. After that drift on, due to the to the yielding of the FA rank inner steel tubes, the sharp increase of residual drift angles observed in all specimens of series A, and the difference in residual drift angles between the five specimens of series A was minimal until R=0.04 rad. From that on, the residual drift angle of A-25F was significantly higher than that of other specimens, indicating that A-25F had been damaged to a certain extent. The residual drift angles of other five specimens continued to increase until R=0.08 rad, and the difference in curves between the five specimens was not significant. The residual drift angles of specimens of series A could be kept below 0.04 rad after being unloaded at R = 0.06 rad.

Compared the residual drift angles of specimens in series B and series C as shown in **Fig. 3-12**, no obvious difference was observed in the measured residual deformation among specimens of series B/C till drift angle reached 0.015 rad as the FB/FC rank inner steel tube began to yield. Afterwards, the residual deformation of the specimens of series B and C began to significantly increase and showed differences on values. After R=0.015 rad, C-25V exhibited significantly higher residual drift angle than other specimens, indicating irreparable damage to the concrete and inner steel tube of it. After R=0.02 rad, the residual deformation of the specimens encased with a FB rank inner steel tube were relatively small. The residual drift angles of specimens of series B/C could be kept below 0.04 rad after being unloaded at R = 0.05 rad.

3.3.3 Equivalent Viscous Damping Coefficient



Fig. 3- 13 Definition of heq



The definition and calculation of the equivalent viscous damping coefficients (h_{eq}) was shown in **Fig. 3-13.** The h_{eq} of all specimens in series A are shown in **Fig. 3-14**. It is noteworthy that the measured showed h_{eq} constant values before R=0.015 rad, and commenced increasing sharply from R=0.02 rad, where the inner steel tubes yielded. This phenomenon implied that

the yielding of inner steel tubes triggered the increase in energy dissipation capacities of SC columns encased with FA rank inner steel tubes. It can also be seen from the results shown in **Fig. 3-14** that the larger the axial load ratio, the larger the measured h_{eq} tended to be.

According to Fig. 3-15, The h_{eq} of all specimens in series B were displayed. It could be seen that the measured showed h_{eq} constant values before R=0.015 rad, the energy dissipation capacity of series B specimens were almost the same. After that with the yielding of inner steel tube, h_{eq} increased sharply from R=0.02 rad. At R=0.02 rad, the specimen B-25V terminated loading due to irreversible large deformation failure. From R=0.02 rad an on, the h_{eq} of the three specimens encased with FB rank inner steel tube rapidly increased, with the highest h_{eq} of B-15VR and the lowest h_{eq} of B-15F. From this result, it can be seen that the specimens with a FB rank hollow inner steel tubes were more prone to have a higher h_{eq} , and infilling concrete into inner steel tube could reduce the h_{eq} and irreparable damage to the specimens. Moreover, it also could be seen that the larger the axial load ratio, the larger the measured h_{eq} .

The h_{eq} of all specimens in series C are shown in **Fig. 3-16.** It could be seen that the measured showed h_{eq} constant values before R=0.01 rad, after that the h_{eq} of series C specimens increased sharply due to the yielding of inner steel tubes. From R=0.01 rad and on, the h_{eq} of specimens in series C began to surge, and be kept around 0.25 before R reached 0.03 rad. The h_{eq} of the specimens C-25V and C-15V were higher than that of the specimens C-25F and C-15F, it could be indicated that for the specimens with a FC inner steel tube, compared to the specimens which infilled concrete into the inner steel tube, the specimens with a hollow inner steel tube were more likely to significantly increase the h_{eq} of the SC columns. On the other hand, the h_{eq} of C-25V and C-25F was greater than that of C-15V and C-15F respectively, which further indicated that increasing the axial compression ratio can increase the h_{eq} of the SC columns.

3.3.4 Strains measured in steel tubes and rebars

For the specimens of series A, the strain profiles of the axial strain of inner tubes and those of the lateral strain of outer tubes along the column height are depicted in **Fig. 3-17**. Only the



strains measured on the initial tensile flange are shown in **Fig. 3-17**. The vertical black chain lines superimposed represent the respective yield strains of the steel tubes.



Fig. 3- 17 Strain distribution along with height of specimens in series A

For the inner steel tubes of the specimens in series A, the axial strains developed within the end region of 300mm, especially at heights of 30mm and 150mm from the bottom of the column, the axial strain values were at their maximum. The axial strains near the column bottom reached yield strain at R=0.015 rad for all specimens of series A. Afterwards, as the height increased, the strain values decreased gradually, and the axial strains of all specimens in series A were almost zero at a height of 600mm from the bottom of the specimens.

For the outer steel tubes of the specimens in series A, the lateral strains concentrated in the region of 150mm from the column bottom, especially at a height of 30mm from the bottom of the column, the axial strain values were at the highest. The horizontal strains of specimens in series A were beyond the yield strain at R=0.01 rad for the specimens subjected to axial compression. The horizontal strain of outer steel tube of specimen A-00F exceeded the yield strain at R=0.015rad. Because specimen A-00F was not subjected to axial compression, the lateral dilation of concrete in this specimen was much less than that in the specimens subjected

to axial compression. As the height increased, the strain value decreased, and the horizontal strains at positions 300mm and 600mm away from the bottom of the specimens were much smaller than the strain values near the bottom of the columns.





Fig. 3- 18 Strain distribution along with height of specimens in series B

For the inner steel tubes of the specimens in series B as shown in **Fig. 3-18**, the axial strains developed within the end region of 150 mm. For the specimen B-15F, the axial strains near the column bottom reached yield strain at R=0.015 rad, and the other three specimens of series B all yielded at R=0.01 rad. This is because the interior of the specimens in series B, except for B-15F, was a hollow square steel tube, without the interaction between infilled concrete and covered concrete. Therefore, the horizontal confinement of the hollow inner steel tubes was smaller, resulting in greater lateral deformation, and the inner square steel tubes were easier to reach yielding. Afterwards, as the height increased, the strain values decreased gradually.

For the outer steel tubes of the specimens in series B as shown in **Fig. 3-18**, the lateral strains concentrated in the region of 30mm from the column bottom, and were beyond the yield strain at R=0.01 rad. At heights of 150mm and 300mm from the upper surface of the stub, the lateral strains tended to approach 0.





Fig. 3- 19 Strain distribution along with height of specimens in series C

For the inner steel tubes of the specimens in series C as shown in **Fig. 3-19**, the axial strains developed within the end region of 150 mm. The axial strains near the column bottom reached yield strain at R=0.01 rad for the specimens C-15F, C-15V and C-25V, only the inner steel tube of C-25F reached yield at R=0.015 rad. The axial strain values of the inner steel tube of series B specimens gradually decreased as the height increased. The lateral strains of the outer steel

tubes at heights of 150mm and 300mm from the upper surface of the stub were very small, and there was almost no deformation of the outer steel tubes at this region.

3.4 Conclusions

In order to obtain fundamental information on the seismic performance of square steel tube encased concrete columns confined by welded circular thin steel tubes, a total of thirteen specimens with different grades of inner square steel tubes were tested under cyclical reversed horizontal load and constant axial load. From the experimental results described in this chapter, the following conclusions can be drawn:

- (1) For the SC columns with a FA rank square inner steel tube encased, sufficient load-carrying capacity and high ultimate drift ratio up to 0.04 rad could be achieved even without infilling concrete and under axial load ratio of 0.25. For the specimens with FB rank square steel tube encased, the effects of infilling concrete and the existence of longitudinal rebars became significant compared to series-A specimens. Under axial load ratio of 0.15, without infilling concrete, the SC columns with FB rank steel tube encased could exhibit ultimate drift ratio of 0.03 rad. On the other hand, when the axial load ratio increased to 0.25, the SC columns with vacant FB rank steel tube showed abrupt drop in the lateral resistance near the drift level of 0.02 rad. For the SC columns with C rank square tube encased, to ensure an ultimate drift angle beyond 0.02 rad, the infilling of concrete was indispensable.
- (2) The higher the axial compression, the larger the ultimate lateral load-carrying capacity of the SC columns because the confinement effect by outer steel tube became more significant along with the axial load level. But the higher axial compression tended to cause the rupture of the welding seam of the outer steel tube at smaller drift level, and hence careful attention must be paid during the process of welding.
- (3) With concrete being infilled, the SC columns with FC rank steel tube encased could exhibit an ultimate drift angle of 0.02 rad even under relatively high axial compression with axial load ratio of 0.25. To ensure sufficient ultimate drift angle to the SC columns under axial

load level up to 0.25, it is recommended to encase at least B rank steel tube with concrete infilled.

(4) Due to the high confinement efficiency of the circular thin steel tube, the maximum lateral resistances of almost all specimens exceeded the calculated results by the AIJ SRC standards and AIJ CFT design guideline. Therefore, to conduct rational design of the proposed SC column section, confinement effect by the circular thin steel tube should be taken into consideration.

References

- [3.1] Architectural Institute of Japan (2014). *AIJ Standard for Structural Calculation of Steel Reinforced Concrete Structures*. (in Japanese)
- [3.2] Architectural Institute of Japan (2008). Design Recommendation for Concrete Filled Steel Tubular Structures. (in Japanese)

CHAPTER FOUR Structural Performance of Steel Tube-Encased Concrete Columns Confined by Bolted Circular Thin Steel Tube

4.1 Introduction

Based on the experimental results of the axial compression tests in Chapter two, the good confinement effectiveness and ductility of the bolted circular thin steel tube has been verified. Furthermore, according to the observations made in Chapter three, it was founded that the specimens with FA rank and FB rank square inner steel tubes encased, and confined by welded circular thin steel tubes, could sustain sufficient load-carrying capacity until R=0.05 rad. However, the welded specimens with FC rank inner steel tube encased may experience local buckling in the absence of infilling concrete. As demonstrated, infilling concrete is indispensable for the specimens with FC rank inner steel tube encased. Moreover, the higher axial compression ratio may lead to the rupture of the welding seam of the outer steel tube at smaller drift levels. From the conclusions drawn in Chapter two and Chapter three, it is rational to replace the welded circular thin steel tubes with the bolted thin steel tubes to confine the SC columns.

Objectives of this chapter are: 1) to obtain basic information on the structural performance of circular concrete columns with the square steel tube encased and confined by the bolted circular thin steel tubes, and 2) to verify the effectiveness of the confinement by the bolted steel tubes by comparing the cyclic performance of the SC columns with those of the SC columns confined by the welded circular thin steel tube described in chapter three.

4.2 Experimental program

4.2.1 Outlines of test specimens

To investigate structural behavior of the proposed SC column confined by bolted circular thin steel tube, six specimens were fabricated and subjected to cyclically reversed lateral force and constant axial compression. The details and primary test results are shown in **Fig.4-1** and **Table 4-1**. The specimens were 1/3 scale circular columns with diameter of 300mm, and the shear span of the specimens was 910mm to give a shear span ratio of about 3.0. As illustrated in the **Fig.4-1 (a)**, the height of the loading stub of each specimen was 500 mm, with the inner square steel tube welded to a baseplate (with a thickness of 22mm) placed at the bottom of the stub. A clearance of 6 mm was provided between the outer bolted circular steel tube and the upper surface of loading stub to guarantee that the outer bolted circular steel tube only provides lateral confinement without bearing axial force.



Fig. 4-1 Details of specimens

Specimen	Outer Tube (SS400) Inner Tube (STKR400)	Section Configuration	fc' (MPa)	N (kN)	n (N/N ₀)	eMmax (kNm)	R _{exp} (rad)
B-15F-b-1.6	φ-303.2×1.6 □-150×150×4.5 FB Rank	Ŧ	33.4	465	0.15	129.2	0.028
D 101 0 1.0						-129.8	-0.037
B-15V-b-1.6		ŦŒŦ	33.4	369		109.0	0.019
						-105.5	-0.020
B-25V-b-1.6		Ŧ	35.3	622	0.25	133.6	0.019
						-127.3	-0.020
B-25V-b-2.3	<i>φ</i> -304.6×2.3 ☐-150×150×4.5 FB Rank	Ŧ	33.9	636		133.0	0.020
						-127.8	-0.020
C-25F-b-1.6	<i>φ</i> -303.2×1.6 □-175×175×4.5 FC Rank	ŦŒŦ	33.4	870		203.0	0.100
						-175.1	-0.065
C-25F-b-2.3	<i>φ</i> -304.6×2.3 ☐-175×175×4.5 FC Rank	Ŧ	33.6	873		197.5	0.090
						-197.9	-0.062

Table. 4-1 Details and primary test results of specimens

Note: $f_c'=$ concrete compressive strength; N= applied axial force; $N_o=f_c'A_c+f_{yt}A_{st}$; $A_{c=}$ sectional area of concrete; $A_{st}=$ sectional area of inner tube; ${}_eM_{max}=$ experimental maximum bending moment; $R_{exp}=$ drift angle at ${}_eM_{max}$

The experimental variables included axial load ratio, infilling of concrete into the encased square steel tube, the grade of the encased square steel tube (FB rank and FC rank), and the thickness of bolted circular thin steel tube (with outer diameter-to thickness ratio of 189 and 132, respectively). The axial load ratio represents the ratio of applied axial force (N) to the nominal axial capacity (N_o) of SC columns section. As shown in the note of **Table 4-1**, the nominal axial capacity No was calculated by simply summing the axial capacities of the encased (inner) steel tube and the concrete portion. The encased inner steel tubes of FB rank and FC rank represent moderate ductile and low ductile steel tubes, respectively, and have square crosssections with a thickness of 4.5mm and outer widths of 150mm and 175mm, respectively. As one can see from **Table 4-1**, of four specimens with inner steel tube of FB rank, no concrete was filled into the inner steel tubes to investigate the effect of in-filled concrete on the structural performance.

As shown in the **Fig.4-1** (b), the bolted circular thin steel tube used for confining concrete was fabricated by at first bending two flat steel plates with target thickness into two semi-circular shapes, and clamping them with two thick steel plates (41mm in width and 9mm in thickness) along the length of the connection flange of the semi-circular steel plates, and then connecting the two pieces of semi-circular steel plates and thick steel plates with high-strength bolts of M12. Due to the potential difficulty associated with assembling the full-length bolted circular thin steel tube and its interference with the lateral force loading apparatus, the uppermost 300mm portion of the test specimens was left unconfined by the bolted steel tube.

4.2.2 Material properties of the steels used

The specimens were fabricated by using ready-mixed concrete and coarse aggregates with a maximum particle size of 20mm. Based on the test results of three cylinders (100mm in diameter and 200mm in height), the average values of 28-days compressive strength, young's modulus and peak strain of the concrete were 34.33MPa, 29.3GPa, and about 0.2%, respectively. The mechanical properties of the encased steel tube and the bolted circular thin steel tube are shown in **Table.4-2**. In **Table.4-2**, the yield strain ε_y is calculated through dividing the yield stress by the Young's modulus. For the FB rank inner steel tube, the yield strain was 0.0018, while for the FC rank inner steel tube, it was approximately 0.0021.

Name	Shape	E _S (GPa)	fy (MPa)	ε_y	f _u (MPa)					
FB-PL4.5	□-150×150	195	351*	0.0018	446					
FC-PL4.5	□-175×175	190	398*	0.0021	477					
PL1.6	<i>ф</i> -303.2×1.6	201	391	0.0019	460					
PL2.3	φ-304.6×2.3	200	348	0.0017	435					

Table. 4- 2 Mechanical properties of steels

Note: E_S : Young's modulus; f_y : Yield stress; ε_y : Yield strain; f_u : Tensile stress; *: based on 0.2% offset method.

4.2.3 Test setup and loading program



Fig. 4- 3 Loading program

As illustrated in the **Fig.4-2**, cyclical reversed horizontal force and constant axial compression were applied to all specimens via hydraulic jacks. A vertical hydraulic jack with a capacity of 1000 kN and connected to a stiff loading frame via a linear slider, was utilized to apply constant axial compression. Simultaneously, a 500kN capacity jack was adopted to apply the cyclically reversed lateral force. The cyclic lateral force was controlled by the drift angle R, which is defined as the ratio of the lateral tip displacement to the shear span. The targeted loading program is shown in **Fig.4-3**. Two complete loading cycles were applied at each level of the targeted drifts (0.005 rad, 0.01 rad, 0.015 rad, and 0.02 rad), and one cycle was applied at each level of the targeted drift after drift angle exceeded 0.02 rad.

4.2.4 Instrumentation and measurement

The locations of biaxial strain gauges for the tested specimens and displacement transducers (DTs) are illustrated in **Fig.4-4** and **Fig.4-5**. Each specimen was equipped with a total of eighteen strain gauges. The strain gauges were located at the column sections 30mm, 150mm, and 300mm from the top of the stub. For the inner square steel tube, strain gauges were embedded on all four surfaces, while for the bolted circular thin steel tube, strain gauges were affixed only to both sides in the loading direction (east and west sides).

As displayed in **Fig. 4-5**, four DTs were used to measure the lateral displacement and the axial shortening or elongation of specimens. The average of the displacements measured by DTs No. 1 and No. 2 were adopted to present the lateral tip displacement for all specimens. The other two DTs (No.3 and No.4) were installed to track the vertical displacement of the specimens, and the average values of their measurements were taken as the axial deformation results.



Fig. 4-4 Strain gauges' position



Fig. 4- 5 DT's position

4.3 Observed behavior and results

4.3.1 Lateral force-drift angle relationships

Fig.4-6 shows the relationship between the horizontal force(*V*) and the drift angle (*R*) of the specimens. The red solid points and green square marks in the figure represent the ultimate horizontal force of specimens and the tensile yield of inner steel tubes, respectively. The red solid lines and blue dotted lines represent the P- Δ mechanism lines based on the ultimate flexural strengths *_{SRC}M_u* calculated by the standards of AIJ [4.1] for general SRC columns and the full plastic ultimate strength *M_p* by the Design Recommendations of AIJ for concrete filled steel tubular (CFT) columns [4.2], respectively (detailed in **Chapter five** below).





For the specimen B-15V-b-1.6, the tensile flange of the square encased steel tube reached its yield strength at a drift angle of 0.01 rad, and the square encased steel tubes of other five specimens reached their yield strength at a drift angle of 0.015 rad. The hysteresis loops were shown an inverted S-shape, and the residual deformation angle was minimal until R reached 0.015 rad. Within this range of drift angles, the primary lateral resistance was provided by the concrete. From R=0.015 rad on, the hysteresis loops became rounder as the inner steel tube yielded, indicating high energy dissipation capacity. The loading for specimens B-25V-b-1.6 and C-25F-b-2.3 was terminated at R=0.09 rad, while other four specimens were loaded up to R=0.1 rad due to limitations imposed by the displacement meter and the apparatus.

All bolted circular thin steel tubes in the specimens under relatively high axial load did not rupture until the drift angle was about 0.09 rad, it is indicated that bolted circular thin steel tube could provide sufficient confinement effect to concrete, enabling the SC columns exhibit high ductility up to large drift angles. For the specimens with a hollow steel tube, a noticeable bulge was not observed until R=0.08 rad. For the specimens with concrete filled into the inner tubes,

only a minor bulge was observed even in a large deformation range at the bottom of bolted circular thin steel tubes. The higher the axial compression, the higher the maximum lateral resistance.

For Specimens infilled with concrete into encased square steel tube:

As shown in **Fig.4-6**, the specimens B-15F-b-1.6, C-25F-b-1.6 and C-25F-b-2.3, which were infilled with concrete into the encased square steel tube, reached their maximum lateral force at a drift angle of approximately 0.03 rad. Notably, when the axial force ratio (n) was 0.25, the maximum lateral forces of the specimens C-25F-b-1.6 and C-25F-b-2.3 were significantly higher than those of B-15F-b-1.6. Subsequently, the lateral resistance of specimens infilled with concrete into encased square steel tube gradually declined, remaining above 70% of the peak load-carrying capacity until R=0.09 rad.

Effect of the thickness of bolted circular steel tubes:

By comparing structural behaviors of specimens C-25F-b-1.6 and C-25F-b-2.3 in **Fig.4-6**, it was worth noting that the difference between their maximum lateral resistance in push direction and their hysteresis loops was not readily apparent, indicating that increasing the thickness of the bolted circular thin steel tube had minimal impact on the positive lateral resistance of specimens infilled with concrete into square encased steel tube.

For Specimens with a hollow encased square steel tube:

According to the **Fig.4-6**, for the specimens B-15V-b-1.6, B-25V-b-1.6, and B-25V-b-2.3 with hollow encased square steel tubes, they reached their maximum lateral force at R=0.02 rad. After the peak load-carrying capacity point, the lateral resistance of these three specimens began to noticeably decrease, especially in the case of the specimen B-25V-b-1.6.

Effect of the thickness of bolted circular steel tubes:

By comparing specimens B-25V-b-1.6 and B-25V-b-2.3, their maximum load-carrying capacity was almost the same, and they exhibited similar hysteresis characteristics until R=0.02 rad. It can be inferred that the thickness of the outer bolted thin steel tube has little impact on

the maximum lateral resistance at R=0.02 rad. From R=0.02 rad and on, it could be seen that the load-carrying capacity of the specimen B-25V-b-1.6 declined rapidly and the reducing speed was much faster than the mechanism lines. Whereas the rate of decrease in peak lateral resistance of the specimen B-25V-b -2.3 was parallel to the mechanism lines. According to these experimental results it could be reflected that increasing the thickness of the outer bolted thin steel tube could effectively improve the seismic performance of SC columns, and mitigate the degradation of lateral resistance at large drift angle. Sufficient load-carry capacity was maintained until R=0.06 rad.

Effect of the axial load ratio:

By comparing specimens B-15V-b-1.6 and B-25V-b-1.6, it could be seen that there was a significant difference in the hysteresis curves between the two test specimens, and the maximum load-carrying capacity of B-25V-b-1.6 was 1.19 times that of B-15V-b-1.6. According to this phenomenon it was observed that increasing the axial force ratio could improve the maximum lateral resistance of the specimens encased with a hollow inner steel tube. Moreover, the hysteresis loop area of B-25V-b-1.6 was fuller and had a better energy dissipation capability than that of B-15V-b-1.6. However, it was noteworthy that the load-carrying capacity of B-25V-b-1.6 rapidly descended after the peak lateral load, whereas B-15V-b-1.6 could maintain a descent speed parallel to the P- Δ mechanism lines until the ending of loading.

Fig.4-7 illustrates the vertical displacement versus drift angle relationship of the proposed specimens. Compared to the curves of the specimens with concrete filled into the inner tubes, the vertical compression deformation at R=0 rad for each hysteresis loop of specimen B-25V-b-1.6, was suddenly increased from the targeted drift angle of 0.03 rad, which was consistent with the moment when its lateral resistance suddenly decreased in **Fig.4-6**. The maximum vertical compression deformation reached 8 mm in the hysteresis loop with R=-0.06 rad, whereas the vertical deformation of the other five specimens did not exceed 2mm when in a compressed state. This phenomenon observed in B-25V-b-1.6 could be speculated to the possible occurrence of local buckling in its encased square steel tube, and the concrete began

to collapse after R=0.04 rad. In general, for specimens with the hollow inner steel tube, inward local buckling of encased inner steel tube was more likely to occur, while for specimens with concrete infilled in the inner steel tube, the infilling concrete expansion within the inner steel tube facilitated interaction with the cover concrete, reducing the susceptibility to local buckling.



Fig. 4-7 The drift angle-vertical displacement relationship

4.3.2 Comparison of experimental skeleton curves with specimens confined by welded circular steel tube



To evaluate the seismic behavior of the proposed SC columns with different connection method of outer circular thin steel tubes, a comparison was made between the envelope curves of specimens confined by bolted circular thin steel tube in this Chapter and the specimens confined by welded circular thin steel tube in Chapter three.

For the various material properties and parameters of the specimens in the Chapter three, the encased square steel tubes of FB rank and FC rank, which were the same as those used in this paper, had yield strengths of 383MPa and 377MPa, respectively. The thickness and yield strength of the welded circular thin steel tube were 1.6 mm and 237MPa, and the average values

of concrete compressive strength, young's modulus and peak strain were 41.7MPa, 33.5GPa, and about 0.2%, respectively.

As shown in the **Fig.4-8**, the red solid lines of the square mark are the skeleton curves of the specimens confined by welded circular thin steel tube in the Chapter three, the blue solid lines of the circular mark and the green solid lines of the triangle mark represented the specimens in this chapter confined by bolted circular thin steel tubes with the thickness of 1.6mm and 2.3mm (outer diameter-to-thickness ratio were 189 and 132), respectively. (for the purpose of elucidating the specimens with different connection methods in Chapter three and Chapter four, the designation of the specimens in Chapter three now includes the letter 'w,' indicating confinement by welded circular steel tube, and "b" in the name of the specimens in this chapter means the specimens were confined by bolted circular steel tubes).

From the **Fig.4-8**, it can be seen that for the specimens in this chapter and in chapter three infilled with concrete into the encased square steel tube (except for the specimen C-25F-w-1.6), the peak load was reached at around R=0.03 rad. While in contrast, the specimens with a hollow inner steel tube arrived their maximum lateral resistance at approximately R=0.02 rad. It can be inferred that infilling concrete within the inner steel tube would likely lead to a higher drift angle at which the peak load was reached.

In **Fig.4-8** (a) and **4-8** (b), the four specimens in these two figures with axial load ratio of 0.15 had almost the same initial stiffness before reaching the maximum lateral resistance, and their performance differed after entering the yield stage. For the specimens confined by welded thin steel tubes, such as B-15F-w-1.6 and B-15V-w-1.6, they demonstrated slightly higher lateral forces compared to their counterparts which were confined by the bolted circular steel tube, like B-15F-b-1.6 and B-15V-b-1.6. After R=0.06 rad, the load-carrying capacity of the two specimens in **Fig.4-8** (a) and 4-8 (b) converged at the same point. These differences were due to variations in strength of materials and were not caused by difference in the details of the outer circular thin steel tube. Additionally, all specimens in **Fig.4-8** (a) and 4-8 (b) maintained stable lateral resistance, retaining over 65% of their maximum horizontal force until unloading.

For the specimens with n=0.25 and no concrete infilling into the FB rank encased square steel tube as shown in **Fig.4-8** (c), before the specimens entered the yield stage, the envelope curves of the three specimens basically overlapped, and the initial stuffiness remained consistent. The specimen confined by a welded steel tube with a thickness of 1.6mm exhibited a peak load similar to that of specimens confined by bolted circular steel tube. However, the welding seam of the specimen B-25V-w-1.6 suddenly ruptured after R=0.02 rad and the load-carrying capacity had sharply declined due to the high axial compression ratio. Compared to the specimen B-25V-w-1.6, specimens B-25V-b-1.6 and B-25V-b-2.3 did not exhibit a sudden decrease in load-carrying capacity due to the premature rupture of the outer bolted circular thin steel tube, and maintained over 70% of their peak loads even at R=0.04rad. It can be seen that for the specimens with FB rank hollow inner steel tube, the confinement effect of welded the outer steel tube, and there would be no sudden fracture of the bolted circular steel tube, improving the ductility of the specimens.

Fig.4-8 (d) shows that three specimens with infilling concrete into FC rank encased square steel tube exhibited minimal differences during cyclic loading in the push direction. However, the welding seam of the welded steel tube in C-25F-w-1.6 suddenly ruptured from a drift angle of 0.02 rad, leading to a rapid decrease in lateral resistance and ultimately terminating horizontal loading. In contrast, specimens C-25F-b-1.6 and C-25F-b-2.3 confined by bolted circular thin steel tube were loaded until the drift angle exceeded 0.09 rad without a sudden decrease in load-carrying capacity.

4.3.3 Residual Drift Angle



Fig. 4-9 Residual drift angle of specimens confined by bolted circular steel tube

Fig. 4-9 shows the residual drift angle of the initially push direction measured at each targeted drift level. It is noteworthy that the measured showed residual drift angle constant values before R=0.015 rad, and commenced increasing sharply from R=0.02 rad, where the inner steel tubes yielded. After R=0.02 rad, as the experimental drift angles increased, the residual drift angles of the six specimens gradually showed differences. the residual drift angles of the specimens B-25V-b-1.6, C-25F-b-1.6 and C-25F-b-2.3 were relatively higher than that of the specimens B-15F-b-1.6, B-25V-b-2.3 and B-15V-b-1.6. Especially for the specimen B-25V-b-1.6, after the experimental drift angle was 0.04, its residual drift angle increased sharply and was significantly higher than the other five specimens. Based on the reasons mentioned before, it can be inferred that the local buckling of the inner steel tube of B-25V-b-1.6 caused irreparable deformation, resulting in a significant rise in its residual drift angles.

4.3.4 Equivalent Viscous Damping Coefficient



Fig. 4- 10 Equivalent viscous damping coefficient



Fig. 4-11 Cumulative Dissipated Energy

The equivalent viscous damping coefficients (h_{eq}) of all specimens in this chapter are shown in **Fig.4-10**. It was noteworthy that the measured h_{eq} showed constant values before R=0.015 rad, and commenced increasing sharply from R=0.02 rad, where the encased square steel tubes had yielded. It is implied that the yielding of inner steel tubes prompted the increase in energy dissipation capacities of SC columns. It can also be seen from the **Fig.4-10** (**b**) that before R=0.03 rad, the difference in h_{eq} between six specimens was not significant, but after that they gradually showed an obviously discrepancy in energy dissipation capacities, especially in B-25V-b-1.6. Comparing all six specimens, it can be seen that the h_{eq} of the specimens with a hollow inner steel tube was greater than that of the specimens with a concrete filled inner steel

tube. This meant that the specimens with a hollow inner steel tube had a relatively higher energy dissipation ability after R=0.03 rad, and was more prone to irreparable damage, leading to irreparable residual deformation.

Fig.4-11 illustrates the cumulative dissipated energy ΔW for six specimens, which was the sum of the areas of all hysteresis loops before each targeted drift angle. Analyzing the experimental results from the **Fig.4-11**, it could be seen that there was little difference in cumulative dissipated energy among all specimens until R=0.02 rad. The specimens C-25F-b-1.6 and C-25F-b-2.3 exhibited significantly higher cumulative dissipated energy compared to the others after R=0.02 rad. The distinction in cumulative dissipated energy between the four specimens with FB encased square steel tube was minimal, and that of B-25V-b-1.6 was not very prominent. As a result, it can be inferred that the increased h_{eq} of B-25V-b-1.6 was attributed to the reduced lateral resistance caused by the local buckling of the encased square steel tube, resulting in a smaller W_e (energy dissipation area).



Fig. 4- 12 Comparison of h_{eq} between specimens with different joint methods

Fig. 4-12 shows a comparison of the h_{eq} of the specimens in Chapter three and Chapter four. Through Fig. 4-12 (a) and 4-12 (b), it can be seen that the connection method of the outer thin steel tube did not affect the energy absorption performance of the specimens with infilled concrete into the FB rank inner steel tube. For the specimens with a FB rank hollow inner steel tube, the connection method of the outer circular steel tube had no effect on the energy absorption performance before R=0.03 rad. However, with the yielding of the inner steel tubes, the h_{eq} of the specimen confined by welded circular steel tube was higher than that of the specimen confined by bolted circular steel tube after R=0.03 rad.

According to the **Fig. 4-12 (c)**, it was observed that up to R=0.015 rad, the h_{eq} curves of the three specimens almost completely overlapped, with no significant difference in energy dissipation performance. At R=0.02 rad, with the inner steel tube enter the ing yield stage, the h_{eq} of the specimen B-25V-w-1.6 was higher than that of the other two specimens. This was attributed to the sudden rupture of the welded seam of the welded circular thin steel tube in B-25V-w-1.6, causing irreversible deformation of the entire specimen and resulting in an increase in h_{eq} Afterwards, the h_{eq} of the specimens B-25V-b-1.6 and B-25V-b-2.3 continued to increase, and after R=0.03 rad, the h_{eq} of B-25V-b-1.6 was significantly greater than that of B-25V-b-2.3 due to the local buckling on inner steel tube of B-25V-b-1.6.

According to the **Fig. 4-12 (d)**, it was observed that before R=0.02 rad, the h_{eq} curves of the specimens C-25F-w-1.6, C-25F-b-1.6 and C-25F-b-2.3 were completely coincident. Subsequently, with the yielding of the inner steel tube, the h_{eq} of these three specimens were rapidly increased. Especially the h_{eq} of C-25F-w-1.6 was higher than that of the C-25F-b-1.6 and C-25F-b-2.3, and at R=0.032 rad, the h_{eq} of C-25F-w-1.6 reached 0.26 due to the suddenly rupture of the welding seam of welded circular steel tube, while the h_{eq} of C-25F-b-1.6 and C-25F-b-2.3 only remained at around 0.2. Afterward, the energy dissipation of specimens C-25F-b-1.6 and C-25F-b-2.3 continued to increase until R=0.09 rad, with a minimal difference in the energy dissipation capacity between these two specimens. It was evident that for specimens with infilled concrete into FC rank inner steel tube, the thickness of the outer circular steel tube had a negligible impact on the energy dissipation performance.

4.3.5 Strains measured in steel tubes



Fig. 4- 13 Strain distribution along with height of bolted specimens encased with FB rank inner square steel tube

Fig. 4-13 shows the strain distribution along with height of bolted specimens with FB rank inner square steel tube encased, the strain profiles of the axial strain of inner tubes and those of the lateral strain of outer tubes along the column height are depicted. The vertical black chain lines superimposed represent the respective yield strains of the steel tubes. The strain gauges on the tensile flange of the inner square steel tube in the specimen B-15F-b-1.6 were damaged at heights of 30mm and 300 from the stub surface, resulting in incomplete strain results.

From the axial strain distribution results of the square inner steel tubes in **Fig. 4-13**, it can be seen that the area near the bottom of the specimen B-15V-b-1.6 yielded at R=0.01 rad, while the region near the bottom of columns of the other five specimens entered the yield stage at R=0.015 rad. The axial strains of inner steel tubes developed within the end region of 150mm, especially at heights of 30mm from the bottom of the column, the axial strain values were at their maximum. For the specimen B-15V-b-1.6, the strain values at a height of 300mm from the upper surface of the stub was relatively larger compared with that of other specimens after R=0.01 rad.

From the lateral strain distribution results of the bolted circular outer steel tubes in **Fig. 4-13**, it was observed that the strain values of the bolted circular thin steel tube of these four specimens were mainly concentrated at a height of 30mm from the surface of the stub, and the lateral strains were very small at heights of 150mm and 300mm. The bolted circular thin steel tubes of specimens B-15F-b-1.6, B-15V-b-1.6, and B-25V-b-2.3 yielded at a drift angle of 0.04 rad. The bolted circular thin steel tube of B-25V-b-1.6 did not yield before the drift angle reached 0.06 rad. It was analyzed that the reason for the later yield of the bolted circular thin steel tube of B-25V-b-1.6 was due to local buckling of the inner steel tube at R=0.04 rad, and irreparable deformation and damage to the inner steel tube and internal concrete was occurred, resulting in incomplete confinement effect on concrete by bolted outer circular thin steel tube.


Fig. 4- 14 Strain distribution along with height of bolted specimens encased with FC rank inner square steel tube

Fig. 4-14 depicts the strain profiles of the axial strain of inner tubes and those of the lateral strain of outer tubes along the column height of the bolted specimens with FC rank inner square steel tube encased.

From the axial strain distribution results of the square inner steel tubes in **Fig. 4-14**, it can be seen that the axial strains of the inner steel tubes in the specimens C-25F-b-1.6 and C-25F-b-2.3 was mainly concentrated at the region with heights of 30mm and 150mm from the upper surface of the stub, and this area of the inner steel tube yields at R=0.015 rad. At R=0.02 rad, the position of the inner steel tube near 300mm from the column bottom also entered the yield stage, generated relatively higher axial strain values.

For the lateral strain results of the bolted circular steel tube of the specimen C-25F-b-2.3, it entered the yield stage at R=0.04 rad, and its confinement effect on concrete was fully utilized when a drift angle was 0.04 rad. Meanwhile, For the lateral strain results of the bolted circular steel tube of the specimen C-25F-b-1.6, their strain changes were significantly concentrated at

a height of 150mm from the upper surface of the stub, and the lateral strains of the bolted circular steel tube were generally small before R=0.06 rad, and the bolted circular steel tube of C-25F-b-1.6 did not enter the yield state until the specimen was loaded in push direction forward to R=0.1 rad.

4.4 Conclusions

In order to obtain fundamental information on the seismic performance of square steel tube encased concrete columns confined by bolted circular thin steel tubes, a total of six specimens with FB rank and FC rank inner square steel tubes encased were tested under cyclical reversed horizontal load and constant axial load. From the experimental results described in this chapter, the following conclusions can be drawn:

- (1) The bolted circular thin steel tube with diameter-to-thickness ratio of 189 could provide sufficient confinement effect to concrete and make the SC columns behave in a very ductile manner up to large drift. It is noteworthy that the bolted thin steel tubes in the specimens under relatively high axial load did not rupture until the end of loading at about 0.09 rad, which is the advantage of the bolted thin tubes over the welded thin steel tubes.
- (2) When the diameter-to-thickness ratio of the bolted steel tube was as large as 189, the SC columns with concrete infilled into the inner square steel tubes of FB and even FC ranks exhibited as large ultimate drift ratios (0.04 rad) as those of the SC columns confined by the welded steel tube.
- (3) Confinement by the bolted circular thin steel tube could ensure the SC columns with hollow square steel tube of FB ranks encased an ultimate drift angle of 0.03 rad even the column was under axial compression with axial load ratio of 25%. Increasing the thickness of the outer bolted steel tube had little significant impact on the peak lateral resistance due to the discontinuity near the connecting portion of the bolted steel tube, but could mitigate the

degradation of lateral resistance at large drift for all SC columns with FB rank and FC rank square steel tubes encased.

References

- [4.1] Architectural Institute of Japan (2014). AIJ Standard for Structural Calculation of Steel Reinforced Concrete Structures. (in Japanese)
- [4.2] Architectural Institute of Japan (2008). Design Recommendation for Concrete Filled Steel Tubular Structures. (in Japanese)

CHAPTER FIVE Analysis Method to Evaluate Seismic Behavior of Square Steel Tube Encased Concrete Columns Confined by Circular Thin Steel Tube

5.1 Introduction

Based on the experimental results in Chapters three and Chapter four, the basic experimental data on the structural performance of the proposed SC columns were obtained. It is crucial to accurately analyze and evaluate the structural performance of the proposed SC columns through current codes and numerical analysis methods. According to the previous researches, numerous seismic evaluation methods have been proposed for SRC components, but it is not clear if these methods can be applied to the proposed circular SC columns. Particularly, it is not clear how we should model the encased square steel tube when conducting flexural analysis of the proposed SC columns. Therefore, it is necessary to develop an accurate and reliable analytical method to evaluate overall seismic behavior of the proposed SC columns. This method should be able to take the confinement effect on concrete into account.

The objective of this chapter is to find the reasonable design equation for assessing ultimate flexural strength of the proposed SC column section by comparing the experimental results with the calculated ones by representative code-prescribed equations, and to propose a numerical analysis method to evaluate the overall structural performance of the proposed SC columns.

5.2 Evaluation of ultimate strength by current codes

Table 5-1 summarized the primary experimental parameters along with the main test results, in which these specimens had been described in Chapter three and Chapter four. To verify the accuracy of seismic design code in Japan for evaluating ultimate capacities of the proposed SC columns, calculation results by the provided method were compared with the experimental results in this section.

Specimen	Oi	uter Tube <i>(SPCC)</i> iner Tube	Section Configuration		Rebars		fc' (MPa)	n (N/No)	eM _{max+} (kNm)	
A-15F	4 202 2×1 6						39.2	0.15	161	
A-00F							38.7	0.00	125	
A-25F	φ	303.2×1.6					41.9	0.25	182	
A-15FR	$\frac{120 \times 120 \times 10}{\text{HBL385}}$				4-D6 (SD295)		41.9		167	
A-15VR	Jyt				f_{ys} =370 MPa		41.3	0.15	161	
B-15F							40.0		149	
B-15V	ϕ -303.2×1.6 $150\times150\times4.5$ STKR400 f_{yt} = 383 MPa				—		41.4		125	
B-15VR					4-D13 (SD295) f_{ys} =330 MPa	40.3		0.15	138	
B-25V							42.0	0.25	145	
C-15F	ϕ -303.2×1.6 $175 \times 175 \times 4.5$ STKR400 $f_{yt} = 377$ MPa						42.1	0.15	168	
C-15V						43.8	0.12	131		
C-25F							42.5	0.25	187	
C-25V	-) -						42.6	0.23	156	
Table 5-1 Continued										
Specimen		Outer Tube (SS400) Inner Tube (STKR400)		Section Configuration			fc' (MPa)	n (N/No)	eM _{max+} (kNm)	
B-15F-b-	1.6	<i>φ</i> -303 2×1 6		Ŧ			33.4	<u></u>	129	
B-15V-b-1.6		$\Box -150 \times 150 \times 4.5$ FB Rank $f_{yt} = 351$ MPa		Ŧ			33.4	0.15	109	
B-25V-b-1.6				Ŧ			35.3		134	
B-25V-b-2.3		$ \begin{array}{c} \phi -304.6 \times 2.3 \\ \Box -150 \times 150 \times 4.5 \\ FB Rank \\ f_{yt} = 351 MPa \end{array} $		Ŧ			33.9		133	
C-25F-b-1.6		ϕ -303.2×1.6 \Box -175×175×4.5 FC Rank f_{vt} =398MPa		Ŧ			33.4	0.25	203	
C-25F-b-2.3		$\phi -304.6 \times 2.3$ $\Box -175 \times 175 \times 4.5$ FC Rank $f_{yt} = 398 \text{MPa}$		#			33.6		198	

Table 5-1 Details and primary test results of specimens

Note: $f_c'=$ concrete compressive strength; N= applied axial force; $N_o=f_c'A_c+f_{yt}A_{st}$; $A_{c=}$ sectional area of concrete; $A_{st}=$ sectional area of inner tube; f_{yt} : Yield stress of inner stee tube ${}_{e}M_{max}=$ experimental maximum bending moment in push direction; $R_{exp}=$ drift angle at ${}_{e}M_{max}$

5.2.1 Ultimate bending strength

To find reliable and accurate design method for the ultimate flexural strength of the circular SC column sections, three equations to compute the axial force N versus ultimate moment M_u for the column section (referred to as $N-M_u$ interaction equations) are adopted in this chapter. They are, 1) the equation prescribed in the Standards of AIJ for general SRC columns (referred to as SRC equation) [5.1], 2) the equation recommended in the Design Recommendation of AIJ for CFT columns (referred to as CFT equation) [5.2], and 3) and the modified CFT equation by taking account of confinement effect by the outer circular thin steel tubes.

According the SRC equation, the ultimate strength calculation equation for the concrete section is shown by Eq. (5-1), Eq. (5-2) and Eq. (5-3).

$$cN_u = (\theta_n - \sin\theta_n \cos\theta_n) \frac{cD^2 \cdot c\sigma_{cB}}{4}$$
(5-1)

$$cM_u = \sin\theta_n \cdot cD^3 \cdot \frac{c\sigma_{cB}}{12} \tag{5-2}$$

$${}_{c}\sigma_{cB} = {}_{c}r_{u} \cdot f_{c}' \qquad {}_{c}r_{u} = \begin{cases} 0.85 \\ 0.85 - 0.6 \cdot sA/(cD^{2}/4) \end{cases}$$
(5-3)

Where $_{c}N_{u}$ is the ultimate axial force of concrete, $_{c}M_{u}$ is the ultimate moment of concrete, $_{c}D$ is the diameter of concrete, $_{c}r_{u}$ is the concrete strength reduction factor, for the infilling concrete, $_{c}r_{u}=0.85$, for the covered concrete, $_{c}r_{u}=0.85-0.6_{s}A/(_{c}D^{2}/4)$, $_{s}A$ is the cross-sectional area of inner steel tube, f_{c}' is the concrete cylinder strength. θ_{n}, x_{n} and $_{c}D$ is shown in Fig. 5-1, $\theta_{n}=\cos^{-1}(1-2x_{n1}), x_{n1}=x_{n}/_{c}D$.



Fig. 5-1 Stress distribution at ultimate state for concrete section

The ultimate strength calculation equation for the inner steel tube section is shown by Eq. (5-4), and Eq. (5-5).

$${}_{s}N_{u} = {}_{s}A \cdot f_{yt} \tag{5-4}$$

$${}_{s}M_{u} = \begin{cases} Z_{p} \cdot f_{yt} - \frac{s^{d}}{2} \cdot \left(sNu - sa_{w} \cdot \frac{f_{yt}}{2} \right) & sa_{w} \cdot \frac{f_{yt}}{2} \leq sNu \leq sA \cdot f_{yt} \\ Z_{p} \cdot f_{yt} & -sa_{w} \cdot \frac{f_{yt}}{2} \leq sNu \leq sa_{w} \cdot \frac{f_{yt}}{2} \\ Z_{p} \cdot f_{yt} + \frac{s^{d}}{2} \cdot \left(sNu + sa_{w} \cdot \frac{f_{yt}}{2} \right) & -sA \cdot f_{yt} \leq sNu \leq -sa_{w} \cdot \frac{f_{yt}}{2} \end{cases}$$
(5-5)

Where ${}_{s}N_{u}$ is the ultimate axial force of inner steel tube, ${}_{s}M_{u}$ is the ultimate moment of inner steel tube, ${}_{s}A$ is the cross-sectional area of inner steel tube (as shown in **Fig.5-2**), Z_{p} is the section modulus of inner steel tube, f_{yt} is the yield strength of inner steel tube, ${}_{s}a_{w}$ is the area of inner steel tube web plate, ${}_{s}D$ is the outer length of square inner steel tube, ${}_{s}t$ is the thickness of inner steel tube, ${}_{s}d={}_{s}D-{}_{s}t$



Fig. 5- 2 Stress distribution at ultimate state for inner steel tube section

When calculating the ultimate strength of specimens by the SRC equation, the ultimate strength of the concrete section and the ultimate strength of the inner steel tube section are simply added.

For the CFT equation, the above equations Eq. (5-1) ~ (5-5) are used, cr_u is taken as 1.0. The ultimate strength of concrete section and the ultimate strength of inner steel tube are no longer simply added directly, but are generalized added while keeping the neutral axis of the concrete section and inner steel tube section consistent. (as shown in **Fig. 5-3**)



The modified CFT equation also can be derived by assuming that the stress distribution in the columns section at the ultimate state follows that recommended in the AIJ Design Recommendation for CFT column section as shown in **Fig.5-3**. In the modified equation, confinement effect by the circular steel tubes will be calculated by using Eq. (5-6), which was

proposed by Sun et al [5.3] for the concrete confined by welded circular steel tubes.

$$f_{cc}' = f_p + 4.1f_r = cr_u \cdot f_c' + 4.1 \cdot \frac{2t}{D - 2t} f_{yt}, \quad K = \frac{f_{cc}'}{f_p}$$
(5-6)

Where f_{cc} is the strength of confined concrete, cr_u is the concrete strength reduction factor (taken as 1.0), f_c is the concrete cylinder strength, D, t, and f_{yt} are the outer diameter, the thickness, and the yield strength of the outer steel tube, respectively. The parameter K defined in Eq. (5-6) is an index to measure confinement degree by transverse reinforcement, and generally referred to as strength enhancement ratio. According to Eq. (5-6), for the specimens confined by welded circular thin steel tube, the strength enhancement ratios range between was 1.23~1.26. Moreover, the strength enhancement ratios for the specimens confined by welded circular steel tube range between was 1.49~1.65.

5.2.2 Comparison of the calculated ultimate strength

The comparisons between the maximum moments measured at the bottom section and the calculated ultimate flexural strength are shown in the **Fig. 5-4**, **Fig.5-5**, **Table 5-2**, and **Table 5-3**. The experimental maximum moment contained the P- Δ secondary moment. In **Fig. 5-4** and **Fig.5-5**, the red lines represent the calculated N- $_{SRC}M_u$ interaction curve by SRC equation ($_{SRC}M_u$ in **Table 5-2** and **Table 5-3**), the blue lines mean the calculated theoretical N- M_p interaction curve by CFT equation (M_p in **Table 5-2** and **Table 5-3**), and the green lines express the calculated theoretical N- M_{pK} interaction curve by the modified CFT equation (M_{pK} in **Table 5-2** and **Table 5-3**).

Specime	Experimen tal result	Calculated result by SRC equation		Calculated CFT ed	d result by quation	Calculated result by modified CFT equation	
ns	_e M _{max} (kNm)	_{SRC} M _u (kNm)	$_{e}M_{ m max}/_{SRC}$ M_{u}	M_p (kNm)	$_{e}M_{\mathrm{max}}/M_{p}$	M_{pK} (kNm)	$_{e}M_{\max}/M_{p}$
A-15F	161	108	1.49	124	1.31	136	1.19
A-00F	125	53	2.37	97	1.29	106	1.19
A-25F	182	125	1.46	138	1.32	155	1.18
A-15FR	167	111	1.50	132	1.27	144	1.16
A-15VR	161	106	1.52	128	1.26	140	1.15
B-15F	149	107	1.40	122	1.23	131	1.14
B-15V	125	100	1.25	115	1.09	124	1.01
B-15VR	138	114	1.21	129	1.07	137	1.01
B-25V	145	116	1.25	125	1.16	139	1.05
C-15F	168	128	1.31	143	1.17	152	1.10
C-15V	131	118	1.11	131	1.00	139	0.94
C-25F	187	146	1.28	158	1.18	172	1.08
C-25V	156	128	1.22	136	1.15	149	1.05

Table 5-2 Comparisons of flexural strength for welded specimens



Series A specimens

From **Fig.5-4** it can be observed that the maximum flexural strength for the specimens with FA rank square inner steel tube encased exceeded the calculated flexural strength $_{SRC}M_u$ by SRC equation as well as the full plastic flexural strength M_p by CFT equation. Ignoring confinement effect by the outer thin steel tube, these two calculated flexural strengths underestimated experimental results too conservatively. By taking account of confinement effect by the welded steel tubes, the calculated flexural strength by modified CFT equation also underestimated experimental results. However, for the specimen A-00F, the modified CFT equation could

accurately evaluate its ultimate strength. **Table 5-2** shows and compares quantitatively the experimental and calculated flexural strength for the specimens with FA rank inner steel tube encased. As obvious from **Table 5-2**, the modified CFT equation could more accurately predict the flexural strength of SC column sections than the SRC and the CFT equations. The ratio of the experimental flexural strength to the calculated result by the modified CFT equation varied between 1.15 and 1.19, having a mean value of 1.17 and a standard deviation of 0.018. The ratio of the experimental flexural strength to the calculated result by the SRC equation ranged from 1.49 to 2.37, with a mean value and standard deviation of 1.67 and 0.39, respectively. For the CFT equation, the ratio of the experimental flexural strength to the calculated result by the calculated result were between 1.26 and 1.32, having a mean value and standard deviation of 1.29 and 0.03, respectively.

Series B and C specimens

For the maximum flexural strength for the specimens with FB and FC rank square inner steel tube encased in **Fig.5-4**, it could be seen that the calculated flexural strength $_{SRC}M_u$ by SRC equation underestimated experimental results. However, the full plastic flexural strength M_p by CFT equation underestimated the ultimate strength of the most specimens with FB rank and FC rank inner steel tubes encased, but it could accurately evaluate the ultimate strength of the specimen B-15V and C-15V. The modified CFT equation could accurately predict the flexural strength of the specimens with FB rank and FC rank square inner steel tube encased, but it overestimated the ultimate strength of the specimen C-15V due to the local buckling occurred in C-15V, which prevented the outer circular steel tube from effectively enhancing the compression strength of the concrete.

As shown in **Table 5-2**, the modified CFT equation could more accurately predict the flexural strength of SC column sections than the SRC and the CFT equations. The ratio of the experimental flexural strength to the calculated result by the modified CFT equation varied between 0.94 and 1.14, having a mean value of 1.05 and a standard deviation of 0.06. The ratio of the experimental flexural strength to the calculated result by the SRC equation ranged from 1.11 to 1.40, with a mean value and standard deviation of 1.25 and 0.08, respectively. For

the CFT equation, the ratio of the experimental flexural strength to the calculated result were between 1.0 and 1.23, having a mean value and standard deviation of 1.13 and 0.07, respectively.

Summering the calculated results of all thirteen specimens confined by welded circular thin steel tube in **Tables 5-2**, the ratio of the experimental flexural strength to the calculated result by the modified CFT equation varied between 0.94 and 1.19, having a mean value of 1.10 and a standard deviation of 0.08. Compared to the equation prescribed in the Standards of AIJ for general SRC columns, and the equation recommended in the Design Recommendation of AIJ for CFT columns, the calculated results by modified CFT equation could more accurately estimate the ultimate strength of specimens confined by welded circular thin steel tube.

Specimens	Experimental results	Calculated result by SRC equation		Calculate CFT e	ed result by equation	Calculated result by modified CFT equation	
	_e M _{max} (kNm)	_{SRC} M _u (kNm)	$_{e}M_{\max}/_{SRC}M_{u}$	M_p (kNm)	$_{e}M_{\rm max}/M_{p}$	M_{pK} (kNm)	$_{e}M_{\mathrm{max}}/M_{pK}$
B-15F-b-1.6	129	95	1.36	108	1.19	122	1.06
B-15V-b-1.6	109	88	1.24	101	1.08	114	0.96
B-25V-b-1.6	134	103	1.30	113	1.19	131	1.02
B-25V-b-2.3	133	102	1.31	111	1.20	134	1.00
C-25F-b-1.6	203	140	1.45	151	1.35	174	1.17
C-25F-b-2.3	198	141	1.41	152	1.30	180	1.10

Table. 5-3 Comparisons of flexural strength for bolted specimens





From **Fig.5-5** it can be observed that the maximum flexural strength for all specimens exceeded the calculated flexural strength $_{SRC}M_u$ by SRC equation as well as the full plastic flexural strength M_p by CFT equation. Ignoring confinement effect by the outer thin steel tube, these two calculated flexural strengths underestimated experimental results too conservatively. On the other hand, by reasonably taking account of confinement effect by the bolted steel tubes, the modified CFT equation gave a very satisfactory prediction of the ultimate flexural strength for the circularly confined SC column section.

To better see the difference in the accuracy of the three equations, **Table 5-3** shows and compares quantitatively the experimental and calculated flexural strength for all specimens. As obvious from **Table 5-3**, the modified CFT equation could more accurately predict the flexural strength of SC column sections than the SRC and the CFT equations. The ratio of the experimental flexural strength to the calculated result by the modified CFT equation varied between 0.96 and 1.17, having a mean value of 1.05 and a standard deviation of 0.08. The ratio

of the experimental flexural strength to the calculated result by the SRC equation ranged from 1.24 to 1.45, with a mean value and standard deviation of 1.34 and 0.08, respectively. For the CFT equation, the ratio of the experimental flexural strength to the calculated result were between 1.08 and 1.35, having a mean value and standard deviation of 1.22 and 0.10, respectively. Ignorance of confinement effect by the outer steel tube led to very conservative prediction of the ultimate flexural strength by 22% and/or 34% on average.

5.3 Analysis method for assessing seismic performance of SC columns





Fig. 5-6 Uniaxial compressive stress-strain envelope curve of confined concrete

Constitutive laws of materials are indispensable to the analysis of cyclic behavior of concrete components and ductility. In the case of the proposed SC concrete columns, reliable and accuracy stress-strain model for concrete plays a predominant role in accurately predicting the overall seismic behavior. A reliable stress-strain model (NewRC model) for concrete proposed by Sakino and Sun [5.4] which can take the confinement effect of stirrups into consideration, was applied in this analysis. Definition can be seen as below, (as shown in **Fig.5-6**):

$$Y = \frac{AX + (D-1)X^2}{1 + (A-2)X + DX^2}$$
(5-7)

Where, $X = \varepsilon_c/\varepsilon_{c0}$, $Y = f_c/f_{cc}$, f_c and ε_c are the stress and strain, f_{cc} and ε_{c0} are the stress and strain at the peak, $A = E_c/E_{sec}$, $E_c = (0.69+0.332\sqrt{f_c'}) \times 10^4$ is the Young's modulus of elasticity of concrete, $E_{sec} = f_{cc}'/\varepsilon_{c0}$ is the secant modulus at the point of peak stress, and D is the parameter governing the slope of descending portion of the stress-strain curve. The proposed stress-strain curve model can be applied to predict the flexural behavior of confined concrete columns under axial load and bending moment with moment gradient (shear), and a semiempirical formula to estimate the compressive strength of concrete confined by welded circular steel tubes was proposed as Eq. (5-6) in Section 5.2.1.



Fig. 5-7 Complete stress-strain curve of confined concrete under compression

The cyclic (unloading and reloading) rules of the stress-strain model proposed by Sun et al. are depicted in **Fig.5-7** and will be directly utilized. During unloading at point A (ε_{un} , f_{un}), the unloading curve is assumed to be a parabola with point B (ε_{pl} , 0) as its peak, where the tangential stiffness is zero. The reloading curve is assumed to be a straight line connecting the reloading point B (ε_{pl} , 0) to point C (ε_{un} , $0.9f_{un}$), as shown in **Fig. 5-7**. Based on these assumptions, both the unloading curve (AB curve) and the reloading curve (BD curve) can be defined by equation Eq. (5-8).

$$f_{c} = \begin{cases} f_{un} (\frac{\varepsilon_{c} - \varepsilon_{pl}}{\varepsilon_{un} - \varepsilon_{pl}})^{2} & unloading \\ \left(\frac{0.9f_{un}}{\varepsilon_{un} - \varepsilon_{pl}}\right) (\varepsilon_{c} - \varepsilon_{pl}) & reloading \end{cases}$$
(5-8)

For simplicity, the plastic residual strain ε_{pl} is defined as the abscissa at the intersection of a straight line from point A and with E_c as its slope with the strain axis. Following this simplification, the residual strain ε_{pl} can be calculated by equation Eq. (5-9).

$$\varepsilon_{pl} = \varepsilon_{un} - \frac{f_{un}}{E_c} \tag{5-9}$$

The stress-strain curve for inner steel tube is defined by Eq. (5-10) that can consider the effect of strain hardening.

I

$$\sigma_{s} = \begin{cases} E_{s} \cdot \varepsilon_{s}, (\varepsilon_{s} < \varepsilon_{sy}) \\ f_{sy}, (\varepsilon_{sy} < \varepsilon_{s} < \varepsilon_{sh}) \\ f_{su} + (f_{sy} - f_{su})(\frac{\varepsilon_{su} - \varepsilon_{s}}{\varepsilon_{su} - \varepsilon_{sh}})^{p}, (\varepsilon_{sh} < \varepsilon_{s} < \varepsilon_{su}) \end{cases}$$
(5-10)
$$p = E_{sh}(\frac{\varepsilon_{su} - \varepsilon_{sh}}{f_{su} - f_{sy}})$$

where, σ_s and ε_s are the axial stress and strain of steel, respectively, E_s is the elastic modulus, f_{sy} and ε_{sy} are the yield stress and strain, respectively, ε_{sh} is the initiation strain of strain hardening, f_{su} and ε_{su} are the tensile strength and corresponding strain, respectively, and E_{sh} is the modulus of strain hardening portion (=0.01 E_s). Fig.5-8 displays outline of the relationship.



Fig. 5-8 Stress-strain model of inner steel tube



Fig. 5-9 Unloading and reloading rules for stress-strain curve of inner steel tube

The unloading and reloading models suggested by Kitajima et al. [5.5] are indicated in **Fig. 5**-**9**, which were applied for inner steel tube. There are three cases for unloading and/or reloading as described below:

1) for the unloading or reloading occur at point A, based on the envelop of Menegotto-Pinto model [5.6], the target point C is located on the reversed curve with the point (ε_{mo} ,0) as its origin. The absolute strain ε_{ss} at point C is assumed to be equal to the experienced maximum strain in the initial direction.

2) for the reloading from point D at the unloading curve: the target point will be pointing A, which is the start point of the previous unloading curve.

3) for the unloading from point E on the reloading curve: the start point D of the previous reloading curve will be taken as the target point.

5.3.2 Analytical assumptions

The analytical curves will be calculated by using the finite fiber method and

the basic assumptions as follows:

- (1) The plane section remains plane after bending.
- (2) The concrete does not resist tensile stress.
- (3) The shear deformation is ignored.
- (4) The length of the lumped hinge region is 1.0D (D is the depth or diameter of column section).

(5) The inner square steel tube is replaced by sixteen longitudinal rebars as shown in **Fig.5-11** (a) with the same cross-sectional area as the original tube (There was a little difference between the area of concrete when the inner steel tube was equivalent to longitudinal rebars and the actual situation, but the difference between the final sum of concrete fiber areas and the actual area of concrete was about 1.5cm², which was within an acceptable range of error). And the stress-strain curve of the equivalent rebars is defined by the elasto-plastic model which considered the effect of strain hardening. The stress-strain relation of four D6 rebars and four D13 rebars is completely elasto-plastic.

(6) NewRC model is adopted to define the stress-strain relation for the concrete with Eq. (5-6) defining the compressive strength of concrete confined by outer circular thin steel tube.

5.3.3 Description of analysis method



Fig. 5- 10 Analytical model



Fig. 5- 11 Cross-section model



Fig. 5- 12 Stress and strain distribution of cross-section model

Numerical analysis was conducted to predict the overall structural behavior of all specimens. A numerical model consisting of elasto-plastic spring and rigid body was assumed as shown in **Fig. 5-10**. The lateral force versus drift angle relationship of the circular SC column will be calculated by using the finite fiber method. For the inner steel tube, it is replaced by sixteen longitudinal rebars (see **Fig.5-11 (a)**) with the same cross-sectional area as the original tube. For the concrete section, it is discretized into finite fibers as shown in **Fig.5-11 (b)**, the area of each fiber is approximated as the equivalent area of a rectangular shape for calculation. The strength enhancement rate of infilled concrete and cover concrete was taken as the same *K*. **Fig.5-12** shows the stress and strain distribution of numerical analysis cross-section model.



5.4 Comparison of the experimental results and numerical analysis

Fig. 5- 13 Comparisons between tested and analytical results of specimens with FA rank square inner steel tube encased (*K* by Eq. (5-6))

Fig.5-13 displays comparisons between the measured *V-R* relationships of the specimens with FA rank square inner steel tube encased and the theoretical curves calculated by the numerical analysis method. The red dotted lines are the analytical results, the black hysteresis curves are the experimental results. As shown in the **Fig.5-13**, the initial stiffness of all specimens with

FA rank inner steel tube encased was larger than the experimental results, because the bottom of the column was attached to steel baseplate. The boundary conditions between concrete and steel baseplate was not accurately simulated, and this effect may become more significant in specimen without axial load (A-00F). The calculated curves for the specimens except for A-25F did predict the peak load-carrying capacity fairly well and could trace the overall structural behavior accurately up to drift angle of 0.08rad. For the specimen A-25F, the numerical curves could estimate the load-carrying capacity well and trace the overall structural behavior up to drift angle of 0.04 rad.



Fig. 5- 14 Comparisons of skeleton curves between tested and analytical results of specimens with FA rank square inner steel tube encased

In order to analyze the difference between the experimental results and analytical values that consider confinement effects and those that do not, as shown in **Fig. 5-14**, the skeleton curves between experimental results and two kinds of analytical results were compared. The red solid lines are the analytical results taking calculate K by Eq. (5-6) into account, the blue dotted lines are the numerical results which does not consider the confinement effect by outer circular steel tubes (K=1.0). According to the curves in **Fig. 5-14**, it could be seen that the calculated curves by considering confinement effect agree very well with the test results up to large drift angle. Ignorance of confinement effect by circular thin steel tube tends to greatly underestimate the



experimental curves. Therefore, the analytical results considering the confinement effect of outer circular thin steel tube are reasonable and accurate.



Fig. 5- 15 Comparisons between tested and analytical results of specimens with FB rank and FC rank square inner steel tube encased (*K* by Eq. (5-6))

Fig.5-15 display comparisons between the measured *V-R* relationships of the specimens with FB rank FC rank square inner steel tubes encased, and the theoretical curves calculated by the numerical analysis method. The red dotted lines are the analytical results taking confinement effect into account, the black hysteresis curves are the experimental results. As shown in the **Fig.5-15**, the calculated curves for the specimens with infilling concrete (B-15F, C-15F and C-25F) did predict the peak load-carrying capacity well and could trace the overall structural behavior accurately until ending of loading. For C-25F, due to rupture of the welding seam of welded outer circular thin steel tube, the loading was terminated until *R*=0.03 rad. For the part before *R*=0.03 rad, the analytical results could effectively predict the load-carrying capacity. For the specimen with a hollow inner steel tube encased (B-15V, B-15VR, B-25V, and C-25V), the analytical results could accurately estimate the maximum load-carrying capacity of them. For the specimen C-15V, the calculated curve overestimated the whole experimental results. This can be attributed to that the encased hollow square steel tubes buckle inward at small drift angle, permitting concrete to more easily dilate inward, and reducing the confinement efficiency of the outer steel tube.



Fig.5- 16 Comparison of experimental strain and analytical strain of series B and C specimens

Fig.5-16 shows a comparison between the experimental strain results and analytical strain results of series B and C welded specimens. The blue solid lines represent the experimental

strain values, which were measured by the vertical strain gauges on the tensile side of the inner steel tube at a height of 30mm from the upper surface of the stub. The red dotted lines represent the analytical strain results. The horizontal yellow dotted lines represent the yielded strains. From the comparison results, it could be seen that the analytical strain values of the eight specimens match well with the vertical experimental strain values at a height of 30mm from the surface of the stub. Among them, for the specimens B-15V, B-15VR, B-25V, and C-25V, only the comparison results before R=0.02 rad could be observed because the strain gauges failed around R=0.02 rad.





Fig. 5- 17 Comparisons between tested and analytical results of specimens confined by bolted circular thin steel tube (*K* by Eq. (5-6))

As shown in the **Fig. 5-17**, the initial stiffness of all specimens was larger than the experimental results because the numerical analysis ignores the effect of deformation in the encased square inner steel tube within the bottom loading beams, but the calculated curves for the specimens with concrete being infilled into the inner square steel tubes (specimens B-15F-b-1.6, C-25F-b-1.6 and C-25F-b-2.3) did predict the peak load-carrying capacity fairly well and could trace the overall structural behavior accurately up to drift angle of 0.08rad. For the specimens with encased hollow square steel tube (specimens B-15V-b-1.6, B-25V-b-1.6, and B-25V-b-2.3), the calculated curves traced the *V-R* response very well until drift angle of 0.03 rad, but overestimated the experimental results at larger drift angle than 0.03 rad. This can be attributed to that the encased hollow square steel tubes might deform or buckle inward at large drift angle, permitting concrete to more easily dilate inward, and reducing the confinement efficiency of the outer steel tube.





Fig.5- 18 Comparison of experimental and analytical results (Bolt K1 by Eq. (5-11))

Fig.5-18 display comparisons between the measured *V-R* relationships and the theoretical curves calculated by the numerical analysis method by further assuming that the confinement effect of the bolted tubes is equal to and half of that of the welded tubes, the reason is that according to the test results of the short SC columns under concentric loading in Chapter two, the confinement effect of the bolted circular thin steel tubes varied between 50% and 100% of the welded steel tubes under the same experimental conditions. The lower bound of the strength enhancement ratio (*K1*) of concrete confined by the bolted steel tube can be given by Eq. (5-11).

$$K1 = 1 + 0.5(K - 1), \quad K = \frac{f'_{cc}}{f_p}$$
 (5-11)

The blue lines superimposed in **Fig.5-18** represent the calculated *V-R* curves corresponding to KI defined by Eq. (5-11).

As depicted in **Fig.5-18**, for the specimens B-15V-b-1.6, B-25V-b-1.6, and B-25V-b-2.3 with hollow inner steel tube, the calculated curves by using Eq. (5-11) to compute the strength

enhancement ratio of the bolted steel tube not only can more accurately assess the maximum lateral force, but also trace the overall structural performance up to 0.08 rad.

The above observations imply that the confinement efficiency of the bolted steel tubes for the SC columns with concrete infilled into the inner square steel tube can be taken equivalent to that of the welded ones, and for the SC columns without concrete infilled into the inner square steel tube, taking half of the confinement efficiency of welded thin steel tubes can more accurately simulate experimental results.

5.5 Conclusion

- (1) The modified CFT equation by taking into account of the confinement effect could give an accurate prediction of the ultimate flexural strength of the SC column sections. For the welded specimens (with FA rank, FB rank and FC rank inner steel tubes encased), the ratio of the experimental flexural strength to the calculated result by the modified CFT equation varied between 0.94 and 1.19, having a mean value of 1.10 and a standard deviation of 0.08. Similarly, the ratio of the experimental flexural flexural strength to the calculated result by the modified CFT equation for the bolted specimens varied between 0.96 and 1.17, having a mean value of 1.05 and a standard deviation of 0.08.
- (2) For the proposed SC columns confined by welded circular thin steel tube, although it can be seen from the comparison of the curves between numerical and experimental results that the numerical analysis results overestimated the initial stiffness of SC columns confined by welded thin steel tubes, the peak load-carrying capacity and the overall structural behavior could be accurately predicted and traced using the numerical analysis method present in this chapter.
- (3) The overall structural performance of the proposed SC columns confined by bolted circular thin steel tube could be reliably and accurately predicted using the numerical analysis method presented in this chapter combining with the assumptions that the confinement

efficiency of the bolted tubes is equal to that of the welded tubes for the columns with concrete filled into the inner tubes, but half of the welded tube for the columns with hollow steel tube encased, respectively.

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CHAPTER SIX Conclusions and Future Works

6.1 Conclusions

In order to promote the application of the proposed SC columns into practice, numerical and analytical works were conducted on the following aspects in this doctor dissertation: 1) to obtain more experimental information on structural performance of the square steel tube encased concrete columns confined by circular thin steel tubes, 2) to investigate the effects of the grades of square encased steel tubes (FA rank, FB rank, and FC rank) on structural performance of the proposed SC columns, 3) to study the influence of the infilling concrete within the inner steel tube, 4) to compare and analyze the confinement effect of circular thin steel tubes with different thicknesses and joint methods, and 5) to find the reasonable design equation for assessing ultimate flexural strength of the proposed SC column section by comparing the experimental results with the calculated ones by representative code-prescribed equations, and to propose an numerical analysis method to evaluate the overall structural performance of the proposed SC columns.

This doctor dissertation consists of six chapters. Based on the experimental and analytical results summarized in this paper, primary finds obtained from chapter two through chapter five will be summarized below as the conclusions of this doctor dissertation.

Investigating the structural performance of the proposed SC columns consists of encased square steel tube and outer circular steel tubes, welded and bolted in chapter two, the following conclusions can be drawn:

(1) Encasing FA rank square steel tube with concrete infilled could enhance both strength and ductility of concrete and well utilize the confinement effect by the outer circular thin steel tubes, both welded and bolted. Without concrete infilled, local buckling of the encased square steel tube tended to decrease ductility of concrete.

- (2) Confinement by the bolted circular steel tubes were superior to that by the welded thin steel tubes from the perspective of enhancement of ductility of concrete. The bolted thin steel tubes did not rupture until as large strain as about 0.06, ensuring secure confinement effect to concrete. On the other hand, the welded thin steel tubes tended to prematurely rupture along the welding seams, implying that careful attention must be paid to the welding process.
- (3) Confinement efficiency of the welded circular thin steel tubes could be accurately evaluated by the semi-empirical formula (proposed by Sun et al.) if the steel plates meet requirement of the JIS. Confinement effect of the bolted thin steel tubes could also be evaluated by the semi-empirical formula when the D/t ratio was smaller than 78, but the confinement efficiency of the bolted tubes with D/t ratio larger than 111 should be assumed as half of that by welded tubes for conservativeness.
- (4) The stress-strain relationships of concrete in the proposed SC columns could be evaluated by the stress-strain curve model (proposed by Sun et al.), which incorporates the strength enhancement ratio K1 for the concrete confined by bolted thinner steel tube with very good accuracy. In particular, for the SC columns confined by the bolted steel tubes, the calculated stress-strain curves traced the experimental curves very well until axial strain of about 0.06.

According to the experimental results of the square steel tube encased concrete columns confined by welded circular thin steel tubes in chapter three, the following conclusions can be drawn:

(5) For the SC columns with a FA rank square inner steel tube encased, sufficient load-carrying capacity and high ultimate drift ratio up to 0.04 rad could be achieved even without infilling concrete and under axial load ratio of 0.25. For the specimens with FB rank square steel tube encased, the effects of infilling concrete and the existence of longitudinal rebars became significant compared to series-A specimens. Under axial load ratio of 0.15, without infilling concrete, the SC columns with FB rank steel tube encased could exhibit ultimate drift ratio of 0.03 rad. On the other hand, when the axial load ratio increased to 0.25, the SC

columns with vacant FB rank steel tube showed abrupt drop in the lateral resistance near the drift level of 0.02 rad. For the SC columns with C rank square tube encased, to ensure an ultimate drift angle beyond 0.02 rad, the infilling of concrete was indispensable.

- (6) The higher the axial compression, the larger the ultimate lateral load-carrying capacity of the SC columns because the confinement effect by outer steel tube became more significant along with the axial load level. But the higher axial compression tended to cause the rupture of the welding seam of the outer steel tube at smaller drift level, and hence careful attention must be paid during the process of welding.
- (7) With concrete being infilled, the SC columns with FC rank steel tube encased could exhibit an ultimate drift angle of 0.02 rad even under relatively high axial compression with axial load ratio of 0.25. To ensure sufficient ultimate drift angle to the SC columns under axial load level up to 0.25, it is recommended to encase at least B rank steel tube with concrete infilled.
- (8) Due to the high confinement efficiency of the circular thin steel tube, the maximum lateral resistances of almost all specimens exceeded the calculated results by the AIJ SRC standards and AIJ CFT design guideline. Therefore, to conduct rational design of the proposed SC column section, confinement effect by the circular thin steel tube should be taken into consideration.

By analyzing the obtained fundamental information on the seismic performance of square steel tube encased concrete columns confined by bolted circular thin steel tubes in chapter four, the following conclusions can be drawn:

(9) The bolted circular thin steel tube with diameter-to-thickness ratio of 189 could provide sufficient confinement effect to concrete and make the SC columns behave in a very ductile manner up to large drift. It is noteworthy that the bolted thin steel tubes in the specimens under relatively high axial load did not rupture until the end of loading at about 0.09 rad, which is the advantage of the bolted thin tubes over the welded thin steel tubes.

- (10) When the diameter-to-thickness ratio of the bolted steel tube was as large as 189, the SC columns with concrete infilled into the inner square steel tubes of FB and even FC ranks exhibited as large ultimate drift ratios (0.04 rad) as those of the SC columns confined by the welded steel tube.
- (11) Confinement by the bolted circular thin steel tube could ensure the SC columns with hollow square steel tube of FB ranks encased an ultimate drift angle of 0.03 rad even the column was under axial compression with axial load ratio of 25%. Increasing the thickness of the outer bolted steel tube had little significant impact on the peak lateral resistance due to the discontinuity near the connecting portion of the bolted steel tube, but could mitigate the degradation of lateral resistance at large drift for all SC columns with FB rank and FC rank square steel tubes encased.

A reasonable design equation for assessing ultimate flexural strength of the proposed SC column section has been developed and a numerical analysis method to evaluate the overall structural performance of the proposed SC columns has been established in Chapter five, the following conclusions can be drawn:

- (12) The modified CFT equation by taking into account of the confinement effect could give an accurate prediction of the ultimate flexural strength of the SC column sections. For the welded specimens (with FA rank, FB rank and FC rank inner steel tubes encased), the ratio of the experimental flexural strength to the calculated result by the modified CFT equation varied between 0.94 and 1.19, having a mean value of 1.10 and a standard deviation of 0.08. Similarly, the ratio of the experimental flexural strength to the calculated result by the modified CFT equation for the bolted specimens varied between 0.96 and 1.17, having a mean value of 1.05 and a standard deviation of 0.08.
- (13) For the proposed SC columns confined by welded circular thin steel tube, although the numerical analysis results show an overestimation in terms of initial stiffness, the peak load-

carrying capacity and the overall structural behavior could be accurately predicted and traced using the numerical analysis method present in this chapter.

(14) The overall structural performance of the proposed SC columns confined by bolted circular thin steel tube could be reliably and accurately predicted using the numerical analysis method presented in this chapter combining with the assumptions that the confinement efficiency of the bolted tubes is equal to that of the welded tubes for the columns with concrete filled into the inner tubes, but half of the welded tube for the columns with hollow steel tube encased, respectively.

6.2 Suggestions and future works

Due to the constraint of time, there are still several important aspects have not been covered in this dissertation. In order to promote the applications of the proposed SC columns in this doctor dissertation necessary further researches are listed below:

(1) Further Investigation of the confinement effect on concrete by bolted circular thin steel tube Although experimental and numerical analysis have been conducted on the confinement effects of the outer bolted circular thin steel tubes with thicknesses of 1.6mm and 2.3mm, as well as their influence on the structural performance of whole specimens, further research needs to be conducted on the discrepancy in confinement effect between the two different thicknesses of bolt steel tubes by adding more test specimens. Furthermore, it is also crucial to analyze how the deformation and expansion of the connection wings of bolted steel tubes influence the confinement effect. Optimizing the size and quantity of high-strength bolts at the bolted steel tube wings is also a priority.

2) Examination of axial behavior on square cross-section SC short columns confined by square thin steel tube

Chapter two of this doctor dissertation investigated the axial performance of circular-section SC columns. However, in practical engineering, square cross-section columns exhibit excellent flexural behavior, and compared to other shapes, square columns have the advantages of easy fabrication and connection, simplifying the construction process. Therefore, a detailed examination of the performance of SC columns with square cross-sections is necessary. This includes a focus on the confinement effect of square thin steel tubes on concrete, with the aim of optimizing and improving their confinement effectiveness.

(3) Investigation for local buckling of the steel tube

Local buckling, as indicated by the experimental results in Chapters two to four, could reduce the load-carrying capacity and stiffness of the proposed SC columns, significantly impacting overall structural performance. To facilitate robust structural design, a comprehensive understanding of the mechanisms and effects of local buckling is essential. Hence, Parametric studies on various design parameters—such as column dimensions, concrete and steel material properties, axial compression ratios, and loading conditions—are necessary. These studies can help identify key factors influencing local buckling behavior.

List of publications

- [1] Chi ZHANG, Chizuru IRIE, Takashi FUJINAGA, Yuping SUN: Structural Performance of Steel Tubeencased Concrete Columns Confined by Circular Thin Steel Tube. コンクリート工学年次論文集, 44 (2), 391 - 396, 2022年
- [2] 山岡 幸喜,入江 千鶴,藤永 隆,張 弛,川田 侑子,宮川 和明,孫 玉平:薄肉鋼管で横補強し た鋼管内蔵円形コンクリート柱の構造性能. Architectural Institute of Japan,日本建築学会技術報 告集,29 (73),1338 - 1343,2023年10月20日
- [3] Chi ZHANG, Koki YAMAOKA, Takashi FUJINAGA, and Yuping SUN: Structural Behaviors of Steel Tube-Encased Concrete Columns Confined by Bolted Circular Thin Steel Tube. Journal of Advanced Concrete Technology (under review)
Doctoral Dissertation, Kobe University

"Structural Performance and Evaluation of Square Steel Tube Encased Concrete Columns

Confined by Circular Thin Steel Tube", 133 pages

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