Behaviour of steel reinforced concrete columns with high strength materials

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**NANYANG
TECHNOLOGICAL
UNIVERSITY**

**BEHAVIOUR OF STEEL REINFORCED CONCRETE
COLUMNS WITH HIGH STRENGTH MATERIALS**

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2017

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Abstract

High strength construction materials are now attractive for composite steel-concrete construction due to their economic and architectural advantages. However, the current design codes for Steel Reinforced Concrete (SRC) columns are in general only applicable to normal strength materials. It is only allowed in Eurocode 4 (EC4) for concrete class C20/25-C50/60 and steel grades S235-S460 to be used in composite columns. On the other hand, it is permissible in Eurocode 2 (EC2) for higher strength concrete (up to C90/105) while Eurocode 3 (EC3) steel grade up to S690. While there are a lot of published works which focused on SRC columns with normal strength materials ($f_{ck} \leq 50 \text{ N/mm}^2$ and $f_y \leq 460 \text{ N/mm}^2$), research on SRC columns with high strength materials is rather limited. This leads to insufficient test data to establish the validity of using high strength materials in SRC columns in EC4. Therefore, this study is geared to further extend the scope of EC4 to include the usage of high strength materials in SRC columns through numerical and theoretical study.

A comparative study of the current confined concrete models was carried out to identify the most accurate model to be used in design and simulation. An analytical method was then proposed to predict the behaviour of SRC columns, taking into account the effect of strain-compatibility and lateral confinement. Next, a finite element (FE) model was developed to analyse the behavior of SRC columns under concentric and eccentric axial loading by ABAQUS. The FE model was validated with published test data from other researchers. Based on the reliability of the finite element simulation, a series of parametric study were conducted by varying certain variables, i.e. the structural steel strength (up to 690 N/mm^2), concrete strength (up to 90 N/mm^2), volumetric ratio, diameter and spacing of lateral reinforcement, column length and eccentricity ratio. The results were compared with EC4 and the proposed method.

It is shown that the ultimate strengths of SRC columns increased linearly with the increase of material strength for short columns. However, increasing material strength had negligible effect on column strength for slender columns. Ultimate

strength of SRC columns increased by either increasing the diameter and volumetric ratio of hoops or decreasing the spacing of hoops. The diameter, spacing and volumetric ratio of hoops had significant effects on the ultimate strength of column reinforced with high strength steel. The effect of material strength on the column strength was significant for column with smaller eccentricity. For column with higher eccentricity, the effect of material strength was only significant for concrete strength less than 70 Mpa.

The comparison indicated that EC4 was conservative in predicting the bearing capacity of SRC columns with material strength ($f_c \leq 90 \text{ N/mm}^2$ and $f_y \leq 550 \text{ N/mm}^2$). However, EC4 overestimated the ultimate strength of SRC columns with steel strength higher than 550 N/mm^2 . Nevertheless, the proposed method was conservative in predicting the bearing capacity of columns with normal and high strength materials. The scope of EC4 can be further extended to cover a wider range of material strength including concrete strength up to 90 N/mm^2 and yield strength of steel up to 550 N/mm^2 . The proposed method, which considering strain-compatibility and lateral confinement effects, can accurately predict the strength of SRC columns with concrete strength up to 90 N/mm^2 and steel strength up to 690 N/mm^2 .

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List of Symbols

A_a	Area of the structural steel
A_c	Area of concrete
A_{ch}	Area of highly confined concrete
A_{cp}	Area of partially confined concrete
A_{cu}	Area of unconfined concrete
A_s	Area of reinforcement
E_a	Modulus of elasticity of structural steel
E_{cm}	Secant modulus of elasticity of concrete
E_s	Modulus of elasticity of reinforcement
$(EI)_{eff}$	Effective flexural stiffness
G_a	Shear modulus of structural steel
I	Second moment of area of the composite section
I_a	Second moment of area of the structural steel
I_c	Second moment of area of the concrete
I_s	Second moment of area of the reinforcement
K_e	Correction factor
L	Length
M_{Ed}	Design bending moment
$M_{pl,a,Rd}$	The plastic resistance moment of the structural steel
$M_{pl,Rd}$	The plastic resistance moment of the composite section
N_{cr}	Elastic critical force in composite columns
N_{Ed}	The compressive normal force
$N_{pl,Rd}$	The plastic resistance of the composite section
$N_{pl,Rk}$	Characteristic value of the plastic resistance of the composite section
$N_{pm,Rd}$	The resistance of the concrete to compressive normal force
b_c	Width of the composite section
b_f	Width of the steel flange
c_y, c_z	Thickness of concrete cover
e	Eccentricity of loading

f_c	The cylinder compressive strength of concrete
$f_{c,p}$	The compressive strength of partially confined concrete
$f_{c,h}$	The compressive strength of highly confined concrete
f_s	The yield strength of reinforcement
f_u	Tensile strength
f_y	The yield strength of structural steel
h_a	Depth of steel section
h_c	Depth of composite section
h_n	Distance from centroidal axis to neutral axis
s	Spacing centre-to-centre of links
t_f	Thickness of steel flange
t_w	Thickness of the steel web

Chapter 1 Introduction

1.1 Background

1.1.1 General Introduction

A steel-concrete composite column is a composite member subjected primarily to compression and bending. For a composite column, the steel and concrete act simultaneously in resisting the imposed loads through the bonding and friction between their interfaces. In the past few decades, composite columns have been widely used as they combine the advantages of steel and concrete materials to achieve an overall enhancement in strength and stiffness.

Composite columns are classified into two principal categories: (1) Steel reinforced concrete (SRC) sections and (2) Concrete-filled hollow steel sections. Typical composite cross-sections are shown in Figure 1.1. In fully encased sections, the steel section is embedded within concrete, as shown in Figure 1.1(a). Partially encased sections consist of steel I- or H-sections and the concrete filled in the void between the steel flanges, as shown in Figure 1.1(b)-(c). Concrete-filled hollow sections are circular or rectangular, as shown in Figure 1.1(d)-(f).

Figure 1.1 Typical cross-sections of composite columns

1.1.2 Steel Reinforced Concrete Column

SRC column is composed of structural steel, concrete, longitudinal and transverse reinforcement. The typical construction process of SRC columns is shown in Figure 1.2. The internal steel section is erected first, followed by the placement of reinforcement and formwork, thereafter concrete is poured into the formwork to form the SRC column.

Figure 1.2 Construction process of SRC columns

SRC columns have been widely used in many countries due to various advantages i.e. high strength and stiffness, and convenient construction. Generally, SRC columns provide better fire performance in comparison with pure steel columns and the CFT columns because the external concrete cover provides protection for the steel section. Investigations on the fire performance of SRC columns have been conducted by many researchers. It was reported that the fire resistance increased as the concrete cover thickness increases. An additional advantage of SRC column is that the concrete prevents the occurrence of local buckling (C.C. Weng and S.I. Yen, 2002).

1.1.3 High Strength Materials

1.1.3.1 Mechanical Properties of High Strength Materials

The yield strength f_y and the ultimate strength f_u for structural steel are given in Table 1.1 and Table 1.2 respectively in accordance with Eurocode3.

Table 1.1 Material properties for structural steel

Grade	Thickness t (mm)			
	$t \leq 40$		$40 \leq t \leq 80$	
	f_y (N/mm ²)	f_u (N/mm ²)	f_y (N/mm ²)	f_u (N/mm ²)
S235	235	225	215	215
S275	275	265	255	245
S355	355	345	335	325
S420	420	400	390	370
S460	460	440	430	410

Table 1.2 Material properties for High strength steel

Grade	Thickness of the element t (mm)					
	$t \leq 50$		$50 \leq t \leq 100$		$100 \leq t \leq 150$	
	f_y (N/mm ²)	f_u (N/mm ²)	f_y (N/mm ²)	f_u (N/mm ²)	f_y (N/mm ²)	f_u (N/mm ²)
S500	500	590	480	590	440	540
S550	550	640	530	640	490	590
S620	620	700	580	700	560	650
S690	690	770	650	760	630	710

Strength classes for normal concrete are defined as Cx/y, where x and y are the cylinder and cube compressive strength respectively. The properties of normal concrete are given in Table 1.3 in accordance with Eurocode 2 (EC2).

Table 1.3 Material properties of normal strength concrete

Properties of concrete	Concrete class						
	C20/25	C25/30	C30/37	C35/45	C40/50	C45/55	C50/60
f_{ck} (MPa)	20	25	30	35	40	45	50
f_{cu} (MPa)	25	30	37	45	50	55	60
f_{ctm} (MPa)	2.2	2.6	2.9	3.2	3.5	3.8	4.1
E_{cm} (GPa)	29	31	33	34	35	36	37

Table 1.4 Material properties of high strength concrete

Properties of concrete	Concrete class				
	C55/67	C60/75	C70/85	C80/95	C90/105
f_{ck} (MPa)	55	60	70	80	90
f_{cu} (MPa)	67	75	85	95	105
f_{ctm} (MPa)	4.2	4.4	4.6	4.8	5.0
E_{cm} (GPa)	38	39	41	42	44

1.1.3.2 Application of High Strength Materials

Column construction consumes considerable amount of resources particularly in tall buildings and bridges. This is when steel-concrete composite columns are often employed to reduce the column size. Currently, composite columns are constructed using normal strength steel and concrete. Increment of the column capacity by using higher strength materials can lead to many economic benefits to both the builders (reduction in materials and construction cost) and the users of the building (more

useable area). Therefore, high strength construction materials provide a cost effective alternative to traditional materials. Various advantages are listed as follows:

- (1) Higher strength and lower weight
- (2) Reduced member size
- (3) Free up more useable area
- (4) Require less construction materials and construction works
- (5) Reduce the use of space and manpower in handling materials

However, the ductility of material decreases with the increasing of strength. Material brittleness is a major problem for high strength concrete, as shown in [Figure 1.3](#). Local buckling is a major problem for high strength steel. To overcome these problems, the SRC columns come into existence, where the ductility of concrete can be improved and the local buckling of steel section can be prevented or minimized.

[Figure 1.3](#) Stress-strain curves for different concrete class

Today, steel grade up to S460 and concrete class up to C50/60 have been widely used in construction all over the world. Higher steel grades (e.g. S690) and high strength concrete (e.g. C90/105) are also available for structural application for

more than three decades. However, the use of high strength materials is still limited due to the lack of design rules and inadequate field experience in SRC columns.

Nowadays, high strength materials have been used in seismically active countries i.e. the U.S.A, Japan, Korea and China. As material research and manufacturing technology improve, the use of higher strength concrete and steel materials will continue to increase as wider applications are being sought in the construction of modern cities.

1.2 Limitation of Current Design Codes

Current design codes for composite columns are limited to the application of normal strength materials as shown in Table 1.5. AISC 360-16 allows the usage of cylinder strength of normal weight concrete in a range of 21 N/mm² to 69 N/mm² and yield strength of steel up to 525 N/mm² in composite columns. Eurocode 4 (EC4) allows the usage of concrete class C20/25-C50/60 and steel grades S235-S460 in composite columns. However, EC2 for concrete structures design allows the usage of concrete class up to C90/105 and EC3 for steel structures design allows the usage of steel grade up to S690.

Table 1.5 Limitation of material strength in current design codes

Codes	Yield strength of steel (N/mm ²)	Cylinder strength of Concrete (N/mm ²)
AISC 360-16	≤ 525	21 ~ 69
EC4	235 ~ 460	20 ~ 50
EC2	-	≤ 90
EC3	≤ 700	-

SRC columns generally exhibit better ductility and higher buckling resistance as compared to steel columns and reinforced concrete (RC) columns. However, EC4 gives a narrower range of material strength of steel and concrete in comparison to EC2 and EC3. It could be highly due to the following two factors:

(1) lack of test data on SRC columns with high strength materials: In view of previous tests on SRC columns, many of the tests focus on the SRC columns with normal strength concrete and steel (with only a few on high strength material). Therefore, there is insufficient test data to strongly establish the validity of using high strength materials in SRC columns in EC4.

(2) strain compatibility issue between high strength steel and concrete: [Figure 1.4](#) explains why the strengths of both normal strength steel and concrete are utilized effectively: The yield strain of normal strength steel is close to the peak strain of concrete. These two materials are therefore strain-compatible and reach their full strengths when the composite section reaches the ultimate strength. The same plot shows that high strength steel and concrete are not strain-compatible in the sense that high strength concrete reaching its peak strain much earlier than yield strain of high strength steel. This implies that concrete will fail earlier than steel and resulting in a partial utilization of capacities of both the materials. For concrete filled steel section, such strain incompatibility is prevented by the confinement of the exterior steel section. In CFT columns, the more ductile exterior steel section provides confinement that increases both the peak strain and strength of enclosed concrete. For SRC columns, there are four ways to increase the compressive strain: (1) use new type of concrete ECC as an exterior skin of the concrete core; (2) take advantage of confinement from lateral hoops and steel section; (3) take advantage of creep and shrinkage effects; (4) use external fiber-reinforced polymer. The most common approach is to use extra lateral reinforcement to provide adequate confinement for concrete. Therefore, to ensure that the strength of high strength steel can be fully developed, the strain compatibility problem should be considered in the columns design.

Figure 1.4 Stress-strain curves of steel and concrete

Therefore, to further extend the scope of Eurocode 4 to include the high strength materials ($f_c > 50$ MPa and $f_y > 460$ MPa) in SRC columns, more research including experimental investigation and numerical simulation should be carried out to investigate the behaviour of SRC columns with high strength materials.

1.3 Purpose and Scope

The use of high strength materials is growing. Although this improves the structural safety and economy as well as reduces the column size and weight, the current design codes for SRC columns are only applicable for normal strength concrete and steel. For example, EC4 is applicable only for composite columns with normal strength materials. The purpose of this study is to investigate the strength of SRC columns with high strength materials under axial loading through theoretical analysis and numerical simulation; and to understand the structural response and modes of failure of SRC columns. A comparative study of available confined concrete models for RC columns is carried out to determine the most accurate model to calculate the stress and strain of confined concrete in RC columns. Then, a modified confined model for SRC columns is proposed with the consideration of the confinement effect from lateral hoops and steel section.

The design method is proposed based on numerical simulation done by ABAQUS and test data collected from literature which will fill the gap by allowing the design

of SRC columns with concrete cylinder strength up to 90 N/mm^2 and yield strength of steel up to 690 N/mm^2 .

Finite element (FE) package (ABAQUS) is used to model the SRC columns subjected to axial loading (concentric and eccentric loading). The results and published test data are compared to verify the validity of FE models. Moreover, parametric study is conducted to analyse the behaviour of SRC column with high strength materials.

The objectives are:

- (1) To assess the current confined concrete models for RC columns collected from literature to determine the most accurate one;
- (2) To propose a modified confinement model with the consideration of the additional confinement from steel section for SRC columns;
- (3) To propose an analytical model with the consideration of lateral confinement to predict SRC column strength;
- (4) To develop a FE model to analyse the behaviour of SRC columns;
- (5) To conduct a parametric study on SRC columns by varying the geometric parameters, strength of material and loading conditions to study the behaviour of SRC columns;
- (6) To investigate the effect of concrete and steel strength on bearing capacity of SRC columns;
- (7) To investigate the effect of diameter, spacing and volumetric ratio of hoops on the bearing capacity of SRC columns;
- (8) To investigate the effect of slenderness ratio and eccentricity ratio on the bearing capacity of SRC columns;
- (9) To compare the numerical results with predicted values from EC4 to assess the validity of EC4 for SRC column with high strength materials;
- (10) To fill the gap of EC4 by allowing the design of SRC columns with concrete cylinder strength up to 90 N/mm^2 and yield strength of steel up to 690 N/mm^2 .

1.4 Outline

This thesis consists of 7 chapters:

Chapter 1 is the general introduction of the research background, objectives and scope of this study.

Chapter 2 provides a literature review of earlier research works on SRC columns including experimental investigation and numerical simulation.

Chapter 3 gives the detail review of design code EC4. Comparative study of the test data collected from literature and predicted values from EC4 on SRC columns are summarized. Information of strain compatibility method and confined concrete model to evaluate the strength of SRC columns are also provided.

Chapter 4 is the comparative study on current confined concrete models. An accurate model is determined by comparing with published test data. And an analytical method with consideration of lateral confinement and strain-compatibility is proposed to predict the strength of SRC columns.

Chapter 5 illustrates the FE analysis of SRC columns. FE model is developed to simulate the behaviour of SRC columns and verified by published test data which shows a good agreement.

Chapter 6 is a parametric study of SRC columns with a variety of the concrete strength, steel strength, slenderness ratio eccentricity ratio, diameter and spacing of hoops, volumetric ratio of hoops and type of loading. The results are compared with these determined from EC4 approach and the analytical model to access the applicability of these methods.

Chapter 7 provides design examples of SRC column with high strength steel and concrete as well as application of EC4 approach and the modified confinement model.

Chapter 8 is the conclusions and recommendations for future research works.

Chapter 2 Literature Review

2.1 Experimental Investigation on SRC Columns

A series of SRC column tests were carried out by [Steven \(1965\)](#) and [Roderick and Loke \(1974\)](#) with a variation of material strength, dimension of cross-section and length of column. [Anslijn and Janss \(1974\)](#) conducted experimental investigations on SRC columns with steel strength up to 500 N/mm^2 . The effects of material strength, dimension of cross-section and column length on the resistance of SRC columns were studied. It was reported that the behaviour of SRC columns was influenced by material strength and slenderness ratio.

The first experimental database was obtained by [SSRC Task Group \(1979\)](#) where both concentrically loaded and eccentrically loaded SRC columns were reported. Design equations for composite members were proposed to evaluate the behavior of composite members.

Experimental study on SRC beam-column was conducted by [Mirza and Skrabek \(1992\)](#). The variation of the bearing capacity of slender composite beam-column was investigated by varying the material strengths and the dimension of cross-section. It was reported that the slenderness ratio, structural steel ratio and the eccentricity ratio had a significant effect on the strength of slender SRC beam-columns. And the concrete strength only influence SRC column with low slenderness ratio.

Experimental tests were conducted by [Chen et al \(1992\)](#) to study the resistance of SRC columns with varying concrete strength, steel section dimensions and slenderness ratio. The behavior of SRC columns with concrete strength up to 65 N/mm^2 was investigated. It was reported that the strength of SRC columns is influenced by concrete strength and slenderness ratio. Similarly, experimental study on SRC columns was conducted by [Han et al \(1992\)](#) and [Han & Kim \(1995\)](#). The test parameters included the dimensions of steel section, yield strength of steel, steel ratio and eccentricity ratio. Test results showed that the steel ratio and eccentricity ratio had a significant effect on the column strength.

Physical tests of SRC beam-column were carried out by [Mirza et al \(1996\)](#) to study the behaviour of beam-columns and bonding conditions at the steel-concrete interface. It was concluded that the bonding conditions had a little effect on the ultimate strength of SRC beam-columns. Comparison of test results and predicted value using ACI 318 and EC4 approaches were also provided which showed that the ACI 318 and EC4 procedures accurately predict the bearing capacity of SRC beam-columns.

Series of tests were carried out by [Chen and Yeh \(1996\)](#), and [Tsai et al \(1996\)](#) to study the behavior of SRC column with various shapes of structural steel. Four types SRC column with I-shaped, H-shaped, T-shaped and cross-shaped steel sections were investigated, as shown in [Figure 2.1](#). The effects of shape of steel section, longitudinal reinforcement and spacing of lateral hoops on the strength of SRC columns were studied in these experimental tests. It was concluded that columns with H-shaped steel showed better performance than columns with I-shaped steel. Column with cross-shaped steel section showed the best performance of the total four types of SRC columns. Additionally, it was shown that the ultimate strength of SRC columns increased with decrement of the spacing of lateral hoops due to confinement effect.

[Figure 2.1](#) Cross sections of SRC columns

The strength and ductility of SRC columns were investigated using fiber element method combined with experimental studies by [EI-Tawil and Deierlein \(1999\)](#). The effects of structural steel ratio, concrete strength and confinement of concrete by lateral hoops on the strength of SRC columns were studied. The data obtained were compared with the design codes ACI 318. It was concluded that the values predicted by ACI 318 were slightly unconservative for high strength concrete, whereas the values predicted by AISC were overly conservative. Therefore, it was proposed that the ACI 318 should be revised to accurately predict SRC column strength.

A review was conducted on experimental and analytical investigation of SRC columns by [Shanmugam and Lakshmi \(2001\)](#). Test data of most of earlier studies was collected and compiled in this study. A detailed discussion on the bond strength, confinement effect, seismic behavior was also presented. It was reported that intensive research was required on bond strength of steel and concrete and confined concrete.

A comparison of the strength of 150 physical samples of SRC columns from literature with the strength calculated using ACI 318 and EC4 were conducted by [Mirza and Lacroix \(2004\)](#). The columns in the study were subjected to axial loading, combined axial loading and bending, and pure bending. Thereafter, an improvement for ACI procedure was recommended.

A database of composite columns tests has been elaborated to evaluate the AISC procedures by [Dong Keon Kim \(2005\)](#). The database consists of SRC columns, SRC beam-columns, CFST columns and CFST beam-columns. The test data was analyzed and compared with current design provisions available in AISC and EC4. It was concluded that EC4 predicted the strength of composite columns accurately and AISC predicted the capacity of beam-columns very well.

The axial load-capacity of SRC columns with various steel shapes (H-, I-, T- and Cross shaped) and volumetric lateral confinement was investigated analytically by [Chen and lin \(2006\)](#). Stress-strain relationships of concrete in SRC columns were established using confinement factors K_p and K_h for partially and highly confined concrete. The results showed that load capacity of SRC columns could be

accurately estimated by this analytical method. The effects of confinement from lateral reinforcement and various steel shapes were also established which showed that the cross-shaped steel had a higher level of confinement.

Experimental study on how the volumetric transverse steel bars influence cyclic behavior of SRC columns were conducted by [Hoang \(2009\)](#). SRC columns with H-shaped and cross-shaped steel were studied in this research. Test data was compared with the strength predicted by ACI and TaiWan SRC code. The study revealed that the required amount of transverse reinforcement specified by ACI and TW-SRC are overly conservative for SRC columns. And, higher strength of SRC columns can be reached by using steel section with wide flange, and the ductility of SRC columns can be enhanced by adding the amount of transverse reinforcement.

Eccentric axial load capacity of seven SRC columns with steel strength 913N/mm^2 and concrete strength 113 N/mm^2 was investigated by [Kim et al. \(2012\)](#). Fully or partially concrete encasement, spacing of hoops and eccentricity ratio were the test parameters considered. Test results showed that for fully encased columns, the first peak load can be determined by spalling of concrete cover and the load bearing capacity continued to increase if the specimen was well confined by lateral rebars. However, for partially encased columns, the steel flange yield first, followed by spalling of concrete. To examine design codes for SRC columns with high strength steel, test results were compared with design codes. It was concluded that ACI 318 underestimated the resistance of SRC columns and EC4 overestimated the test results.

The strength of SRC columns with various sectional shapes under eccentric loading was also studied by [Kim et al \(2014\)](#). This study focused on three parameters: high strength steel (up to 812 N/mm^2), high strength concrete (up to 184 N/mm^2), and various shapes of steel section (traditional H-shaped and new L-shaped). It was reported that early concrete crushing occurred in the SRC columns. When compared to traditional SRC column, the load capacity of columns with four L-shaped steel significantly increased because of the high level confinement of corner L steel section.

2.2 Summary of Earlier Tests

Experimental investigation on SRC columns was conducted by [Anslijin and Janss \(1974\)](#), [Stevens \(1965\)](#), [Matsui \(1979\)](#), [SSRC Task Group \(1979\)](#), [Roik et al.\(1989\)](#), [Han et al \(1992\)](#), [Mirza et al.\(1996\)](#), [Chen and Yeh \(1996\)](#), [Kim et al.\(2013\)](#) and so on. These tests were conducted on SRC columns with various concrete and steel strength, steel section, reinforcement ratio, slenderness ratio and eccentricity ratio. On the other hand, extensive reviews of most of these tests were covered by [Shanmugam and Lakshmi \(2001\)](#), [Mirza et al.\(2004\)](#) and [Kim \(2005\)](#). As the last database was compiled by Kim D.K. in 2005, there is a need to review and complete it with the latest test data. Therefore, a new compilation of a significant amount of SRC columns was carried out in this study.

234 tests of SRC columns were obtained from literature, including concentric and eccentric loading. Most of the test specimens were SRC columns with normal strength materials, only a few with high strength materials. The dimension and material properties of test specimens are summarized in [Table 2.1](#), where $b \times h$ is the cross-section of SRC columns; f_c is the compressive cylinder strength of concrete; f_y and f_s is the yield strength of structural steel and reinforcement respectively; ρ_{ss} is the steel ratio; ρ_{sr} is the longitudinal reinforcement ratio; e/h is the eccentricity ratio.

Table 2.1 Summary of Dimensions and Material Properties of SRC Test

Properties	$b \times h$ (mm x mm)	f_c (MPa)	f_y (MPa)	f_s (MPa)	ρ_{ss} (%)	ρ_{sr} (%)	e/h
Minimum Value	160 x 160	13	223	276	2.7	0	0
Maximum value	406 x 305	104	913	642	15	3.4	1.5

According to EC4, normal strengths are considered when $f_c \leq 50$ MPa and $f_y \leq 460$ MPa, and high strength is considered when $f_c > 50$ MPa and $f_y > 460$ MPa. [Figures 2.2 and 2.3](#) illustrate the distribution of steel strength and concrete strength in earlier tests respectively. It indicates relatively less SRC column tests with high strength materials, only 7% of the total tests conducted. In these tests, 4% of the specimens are SRC columns with high strength concrete, 4% specimens are SRC

columns with high strength steel and 3% with both high strength concrete and steel. Therefore, test data is insufficient to establish the validity of using high strength materials in SRC columns in EC4. Thus, research on SRC columns with high strength materials is necessary to investigate their behaviour.

Figure 2.2 Distribution of f_y for SRC column tests

Figure 2.3 Distribution of f_c for SRC column tests

2.3 Numerical Simulation on SRC Columns

A FE model was developed using ABAQUS by Ellobody and Young (2011) to analyze the behavior of axially loaded SRC columns. The material properties of individual component and bond behavior between these components were considered in this model. This model was validated by published test data. Finally, a

parametric study was conducted to investigate the effect of concrete strength (30 N/mm², 70 N/mm² and 110 N/mm²), yield strength of structural steel (275 N/mm², 460 N/mm² and 690 N/mm²) and slender ratio (0.2 -1.82) on the SRC column strength. It was observed that for slender SRC columns it has a small effect on increasing strength of SRC columns by increasing yield strength of structural steel. When these were compared with the design codes, it was found that EC4 accurately predicted the resistance of SRC columns with normal strength materials while AISC code was conservative.

On the other hand, numerical simulation of pin-ended SRC columns subjected to eccentric loading acting along major axis was conducted by [Ellobody and Young \(2011\)](#). A parametric study was conducted to estimate effects of concrete strength (30 - 110 N/mm²), yield strength of structural steel (275 - 690 N/mm²) and eccentricity (0.125D, 0.25D, 0.375D) on the SRC column strength. It was reported that for small eccentricity 0.125D, the strength of SRC columns increased significantly by increasing yield strength of structural steel. For high eccentricity 0.375D, increasing steel strength had negligible effect on the strength of SRC columns. Additionally, it was concluded that EC4 accurately estimated the compression resistance, while the bending resistance of SRC columns was overestimated based on the comparison of simulation results and predictions of EC4.

A nonlinear 2-D beam finite element was proposed by [Goncalves and Carvalho \(2014\)](#) to simulate the buckling behavior of SRC columns with I-sections. Parametric study was conducted to assess the influence of member imperfections, concrete strength (C25/30 to C45/55) and steel strength (S235 and S460). It was found that the FE model was very easy to implement and accurately predicted the buckling resistance.

2.4 Summary of Earlier Numerical Simulation

The behavior of SRC columns with high strength steel and concrete has not received the same level of attention as SRC columns with normal strength materials. Although [Ellobody and Young \(2011\)](#) have performed simulations on SRC columns with high strength materials, more research on SRC columns with high strength

materials is required to gain a better understanding of their behavior and to fill the gap in EC4 by allowing the design of SRC columns with concrete cylinder strength up to 90 N/mm^2 and steel yield strength 690 N/mm^2 . So far, no simulation was found in literature highlighting the behavior of SRC columns with higher strength reinforcing bars. Thus further research should be focused on the following aspects:

- (1) Behavior of SRC columns with high strength steel (up to 690 N/mm^2), high strength concrete (up to 90 N/mm^2) and high strength reinforcement (up to 600 N/mm^2);
- (2) Effect of various steel shapes (I-, H-) on the strength of SRC columns with high strength structural steel, concrete and reinforcement;
- (3) Effect of different spacing and diameter of lateral reinforcement on the strength of SRC columns with high strength structural steel, concrete and reinforcement;
- (4) Effect of different diameter and number of longitudinal reinforcement on the strength of SRC columns with high strength structural steel, concrete and reinforcement;

2.5 Confinement Effect on SRC Columns

As mentioned in previous section, high strength steel and high strength concrete are not strain-compatible because high strength concrete reaching its peak strain much earlier than yield strain of high strength steel. This implies that concrete will fail earlier than steel and resulting in a partial utilization of capacities of both the materials. However, the yield strength of the steel section may be developed by the increased concrete strain. The confining pressure from lateral hoops and steel section can improve the ductility and strength of the concrete.

A stress-strain model was proposed by [Mander et al \(1988\)](#) for concrete confined by lateral hoops to predict the strength and strain of confined concrete. The influence of various types of confinement including spiral hoops and rectangular hoops is taken into account by defining the lateral confining pressure. In Mander model, it was assumed that lateral hoops were yielded at the ultimate strain of concrete.

However, it was found that high strength hoops may not yield from experimental investigation carried out by [Sheikh and Uzumeri \(1980\)](#).

An iterative procedure was proposed to predict the real stress of hoops in a stress-strain model which was proposed by [Cusson and Paultre \(1995\)](#). This model was modified by Legeron and Paultre (2003) based on strain compatibility. The modified model is applicable for high strength concrete confined with high strength reinforcing steel. The real stress in lateral hoops could be calculated from a single equation.

The concrete in SRC columns could be divided into three regions: unconfined concrete region (UCC), partially confined concrete region (PCC) and highly confined concrete region (HCC) based on the analysis from Mirza and Skrabek (1992). The additional confining stress from steel section was studied by EI-Twail (1996). Equations for predicting the lateral confining stress from steel section was also proposed by EI-Twail. An analytical model was proposed by Chen and Lin (2005) to predict the behaviour of SRC columns. The ranges of two factors (partially confinement factor and highly confinement factor) are given in accordance with several experimental tests. However, the results are case-dependent, no formula for calculating the strength and strain of HCC is developed.

Besides take advantage of confinement from hoops and steel section, there are some other ways to increase the compressive strain of concrete, i.e. use Engineered cementitious composite (ECC) as an exterior skin of the concrete core; take advantage of creep and shrinkage effects. ECC as a fiber-reinforced high performance material was introduced by [Li et al \(2001\)](#). It can achieve high ductility with ultimate strain as high as 5%. The strain-compatibility issue between high strength steel and concrete could be solved by using ECC in SRC column. The behaviour of RC members with high strength reinforcement SAS 670 was investigated by [Falkner et al \(2008\)](#). The yield strength of SAS 670 high strength reinforcement, whose yield strain exceeds the compressive strain of concrete, can be developed by taking advantage of creep and shrinkage effects to increase the concrete strain. However, there are conditions in which creep and shrinkage are not significant, not high-rise buildings and dead loads are not significant.

In general, the most common and traditional approach is to use transverse reinforcement and steel section to provide adequate confinement for the concrete core to increase the strength and ductility.

For partially confined concrete in SRC columns, the confinement effect can be determined from the expressions for reinforced concrete columns which were extensively studied. Additionally, CEB Model code 1990, EC2 and FIB Model Code 2010 also provide expressions for calculating the strength and strain of confined concrete in RC columns. Therefore, future study should focus on the effect of highly confined concrete (HCC) on the performance of SRC columns.

Chapter 3 Review of Design Codes

3.1 Eurocode 4

EC4 provides two methods (general method and simplified method) to estimate the resistance of composite columns. General method is applicable for composite columns with non-symmetrical cross-section over the column length. Simplified design method is applicable for composite columns with symmetrical and uniform cross-section over the column length. This method is based on some assumptions of the geometrical configurations of composite columns. Additionally, the buckling curves for bare steel are used in SRC column design. According to EN1994-1-1, the limits of applicability of simplified method are given in the following:

- (1) The limitation of steel contribution ratio δ is:

$$\delta = \frac{A_a f_{yd}}{N_{pl,Rd}} \quad (3.1)$$

$$0.2 \leq \delta \leq 0.9 \quad (3.2)$$

- (2) The relative slenderness ratio of SRC column $\bar{\lambda}$ is limited to 2.0.

- (3) For SRC column, the concrete cover thickness must satisfy the following limits:

$$40\text{mm} \leq c_y \leq 0.4b \quad 40\text{mm} \leq c_z \leq 0.3h \quad (3.3)$$

- (4) The depth to width ratio of the column cross-section should satisfy:

$$0.2 < h_c / b_c < 5.0 \quad (3.4)$$

- (5) The maximum amount of longitudinal reinforcement is 6% of the concrete area.

Before calculating the resistance of the composite cross-section, a check on local buckling must be performed. The effect of local buckling may be neglected if the structural steel is fully encased and the concrete cover thickness is adequate.

3.1.1 SRC Columns Subjected to Concentric Loading

The resistance to compression $N_{pl,Rd}$ of SRC column cross-section is obtained from:

$$N_{pl,Rd} = A_a f_{yd} + 0.85 A_c f_{cd} + A_s f_{sd} \quad (3.5)$$

Where A_a , A_c and A_s are the cross-sectional areas of the steel, concrete and reinforcing steel respectively; f_{yd} , f_{cd} and f_{sd} are the design strengths of the steel, concrete and reinforcing steel respectively.

The relative slenderness, $\bar{\lambda}$, is defined as:

$$\bar{\lambda} = \sqrt{\frac{N_{pl,Rk}}{N_{cr}}} \quad (3.6)$$

where:

$N_{pl,Rk}$ is the plastic resistance calculated by:

$$N_{pl,Rk} = A_a f_y + 0.85 A_c f_{ck} + A_s f_{sk} \quad (3.7)$$

N_{cr} is the elastic critical buckling force.

The elastic critical load N_{cr} is given by the following expression:

$$N_{cr} = \frac{\pi^2 (EI)_{eff}}{L_{cr}^2} \quad (3.8)$$

Where $(EI)_{eff}$ is the effective flexural stiffness of SRC column and L_{cr} is the buckling length.

The flexural stiffness $(EI)_{eff}$ of the composite section is calculated by the sum of the flexural stiffness of the structural steel, reinforcement and concrete:

$$(EI)_{eff} = E_a I_a + E_s I_s + K_e E_{cm} I_c \quad (3.9)$$

Where K_e is a correction factor, take as 0.6; I_a , I_c and I_s are the second moments of area of the structural steel, concrete and reinforcing steel respectively; E_{cm} is the elastic secant modulus of the concrete.

For SRC columns in compression, the normal force, N_{Ed} , is required to satisfy the following expression:

$$\frac{N_{Ed}}{\chi N_{pl,Rd}} \leq 1,0 \quad (3.10)$$

where

χ is the reduction factor

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}} \quad \text{but } \chi \leq 1,0 \quad (3.11)$$

and

$$\Phi = 0.5 \left[1 + \alpha (\bar{\lambda} - 0.2) + \bar{\lambda}^2 \right] \quad (3.12)$$

Where α is an imperfection parameter which is dependent on the buckling curve.

The buckling curves for SRC column are shown in [Table 3.1](#). Using [Table 3.1](#) to determine the relevant buckling curve (a, b or c) in the design of SRC columns, and then the parameter α can be determined according to [Table 3.2](#). Thus the value of the reduction factor χ can be determined using based on the above equations.

[Table 3.1](#) Buckling curves and member imperfections

Cross-section	Axis of buckling	Buckling curve	Member imperfection
	y-y	b	$L/200$
	z-z	c	$L/150$
	z-z	b	$L/200$

Table 3.2 Imperfection for buckling curves

Buckling curve	a	b	c
Imperfection factor α	0.21	0.34	0.49

3.1.2 SRC Columns Subjected to Eccentric Loading

The resistance of a SRC column subjected to compression and bending is estimated through an interaction curve. A polygonal diagram instead of the interaction curve is used in the simplified method given in EC4, as shown in [Figure 3.1](#). There are four positions of the plastic neutral axis in the interaction curve. In each position, the expressions for calculating the compressive resistance N_{Rd} and moment M_{Rd} can be determined.

Figure 3.1 Interaction curve of composite columns

The method to determine the point in the interaction curve is to assume a location for plastic neutral axis. Then, the resistance N_{Rd} is calculated by summing the forces of individual component, and the moment resistance M_{Rd} is determined by taking moments of corresponding forces about the centroid of the un-cracked section. Therefore, the interaction curve is determined by moving neutral axis from one point to another, and finding N_{Rd} and M_{Rd} from the corresponding stress blocks.

As shown in Figure 3.1, point A represents pure compression while point B pure bending. The plastic neutral axis at point C is symmetrical to point B. The plastic neutral axis at point D is located in the centre of the cross-section. The maximum moment resistance $M_{\max,Rd}$ is achieved at point D. Figure 3.2 represents the stress distributions of the composite cross-section corresponding to the points A, B, C and D. The significance of each of these points is described as follows:

Figure 3.2 Stress distributions at different position

(1) Point A - Pure Compression

Point A represents pure compression. The bending moment is zero at this point.

$$N_A = N_{pl,Rd}$$

(2) Point B - Pure Bending

Point B represents the moment resistance of the composite section, without compression force:

$$M_B = M_{pl,Rd} = (W_{pa} - W_{pa,n}) f_{yd} + (W_{ps} - W_{ps,n}) f_{sd} + 0.5(W_{pc} - W_{pc,n}) \alpha_c f_{cd} \quad (3.13)$$

Where W_{pa} , W_{ps} , W_{pc} are the plastic section moduli for structural steel, reinforcement and concrete respectively; $W_{pa,n}$, $W_{ps,n}$, $W_{pc,n}$ are the plastic section moduli for the structural steel, reinforcement and concrete within region $2h_n$ respectively.

(3) Point C

Moment resistance is equal to point B. The axial compression force of the SRC column is obtained from:

$$N_C = N_{pm,Rd} = A_c \alpha_c f_{cd} \quad (3.14)$$

$$M_C = M_B = M_{pl,Rd}$$

(4) Point D

The compression and moment resistance of SRC column can be obtained from:

$$N_C = 0.5N_{pm,Rd}$$

$$M_D = M_{max,Rd} = W_{pa} f_{yd} + W_{ps} f_{sd} + 0.5W_{pc} \alpha_c f_{cd} \quad (3.15)$$

After the determination of the four points, the interaction curve can be obtained.

The values of the parameters mentioned in above equations for SRC column sections shown in [Figure 3.3](#) are given in the following:

[Figure 3.3](#) Typical SRC column section

(1) Major -axis

The plastic section modulus of structural steel is obtained from the following equation:

$$W_{pa} = \frac{(h - 2t_f)^2 t_w}{4} + b t_f (h - t_f) \quad (3.16)$$

The plastic section modulus of concrete is calculated from:

$$W_{pc} = \frac{b_c h_c^2}{4} - W_{pa} - W_{ps} \quad (3.17)$$

The plastic section modulus of longitudinal reinforcement is determined by:

$$W_{ps} = \sum_{i=1}^n A_{s,i} [e_i] \quad (3.18)$$

where e_i is the distance of longitudinal reinforcement to the relevant middle line.

For different neutral axis position, h_n and $W_{pa,n}$ are calculated from:

(a) Neutral axis in the web, $h_n \leq h/2 - t_f$

$$h_n = \frac{A_c \alpha_c f_{cd} - A_{sn} (2f_{sd} - \alpha_c f_{cd})}{2b_c \alpha_c f_{cd} + 2t_w (2f_{yd} - \alpha_c f_{cd})} \quad (3.19)$$

$$W_{pa,n} = t_w h_n^2 \quad (3.20)$$

Where A_{sn} is the reinforcement area within the region $2h_n$ and the value of α_c is 0.85.

(b) Neutral axis in the flange, $h/2 - t_f < h_n < h/2$

$$h_n = \frac{A_c \alpha_c f_{cd} - A_{sn} (2f_{sd} - \alpha_c f_{cd}) + (b - t_w)(h - 2t_f)(2f_{yd} - \alpha_c f_{cd})}{2b_c \alpha_c f_{cd} + 2b(2f_{yd} - \alpha_c f_{cd})} \quad (3.21)$$

$$W_{pa,n} = b h_n^2 - \frac{(b - t_w)(h - 2t_f^2)}{4} \quad (3.22)$$

(c) Neutral axis outside the steel section, $h/2 \leq h_n \leq h_c/2$

$$h_n = \frac{A_c \alpha_c f_{cd} - A_{sn} (2f_{sd} - \alpha_c f_{cd}) - A_a (2f_{yd} - \alpha_c f_{cd})}{2b_c \alpha_c f_{cd}} \quad (3.23)$$

$$W_{pa,n} = W_{pa} \quad (3.24)$$

The plastic modulus of concrete within the region $2h_n$ is given by:

$$W_{pcn} = b_c h_n^2 - W_{pa,n} - W_{ps,n} \quad (3.25)$$

The plastic modulus of the total reinforcement within the region $2h_n$ is given by:

$$W_{ps,n} = \sum_{i=1}^n A_{sn,i} [e_{z,i}] \quad (3.26)$$

Where $A_{sn,i}$ is the longitudinal reinforcement area within the region $2h_n$, and $e_{z,i}$ is the distance from the middle line.

(2) Minor axis

The plastic section modulus of the structural steel is obtained from:

$$W_{pa} = \frac{(h - 2t_f)t_w^2}{4} + \frac{2b^2t_f}{4} \quad (3.27)$$

The plastic section modulus of concrete may be calculated from:

$$W_{pc} = \frac{b_c^2 h_c}{4} - W_{pa} - W_{ps} \quad (3.28)$$

The plastic section modulus of longitudinal reinforcements is calculated from:

$$W_{ps} = \sum_{i=1}^n A_{si} [e_i] \quad (3.29)$$

where e_i are the distances of longitudinal reinforcement to the relevant middle line.

For different neutral axis position, h_n and $W_{pa,n}$ are calculated from:

(a) Neutral axis in the web, $h_n \leq t_w/2$

$$h_n = \frac{A_c \alpha_c f_{cd} - A_{sn} (2f_{sd} - \alpha_c f_{cd})}{2h_c \alpha_c f_{cd} + 2h(2f_{yd} - \alpha_c f_{cd})} \quad (3.30)$$

$$W_{pa,n} = h h_n^2 \quad (3.31)$$

(b) Neutral axis in the flange, $t_w/2 < h_n < b/2$

$$h_n = \frac{A_c \alpha_c f_{cd} - A_{sn} (2f_{sd} - \alpha_c f_{cd}) + t_w (2t_f - h) (2f_{yd} - \alpha_c f_{cd})}{2h_c \alpha_c f_{cd} + 4t_f (2f_{yd} - \alpha_c f_{cd})} \quad (3.32)$$

$$W_{pa,n} = 2t_f h_n^2 - \frac{t_w^2 (h - 2t_f)}{4} \quad (3.33)$$

(c) Neutral axis outside the steel section, $b/2 \leq h_n \leq b_c/2$

$$h_n = \frac{A_c \alpha_c f_{cd} - A_{sn} (2f_{sd} - \alpha_c f_{cd}) - A_a (2f_{yd} - \alpha_c f_{cd})}{2h_c \alpha_c f_{cd}} \quad (3.34)$$

$$W_{pa,n} = W_{pa} \quad (3.35)$$

The plastic modulus of concrete within the region $2h_n$ is given by:

$$W_{pc,n} = h_c h_n^2 - W_{pa,n} - W_{ps,n} \quad (3.36)$$

The plastic modulus of the total reinforcement within the region $2h_n$ is given by:

$$W_{ps,n} = \sum_{i=1}^n A_{sn,i} [e_{y,i}] \quad (3.37)$$

3.2 Comparison of Test and Design Codes

3.2.1 SRC columns with normal strength material

Experimental investigation on SRC columns was conducted by Anslijin and Janss (1974), Stevens (1965), Matsui (1979), SSRC Task Group (1979), Roik et al.(1989), Han et al (1992), Mirza et al.(1996), Chen and Yeh (1996) and so on. These tests were conducted on SRC columns with various concrete and steel strength, steel section, reinforcement ratio, slenderness ratio and eccentricity ratio. On the other hand, extensive reviews of most of these tests were covered by Shanmugam and Lakshmi (2001), Mirza et al.(2004) and Kim (2005). The strengths of 207 SRC column tests with normal strength materials ($f_c \leq 50$ MPa and $f_y \leq 460$ MPa) were collected from the above published research papers. The predicted values of these 207 test specimens were also determined from the EC4 approach. The test results were compared with the calculated results to obtain the non-dimensional ratios. The summary of the comparison is shown in [Table 3.3](#).

It is shown that the mean value from experimental to calculated resistance using EC4 procedure is 1.21, with a standard deviation 0.20 and coefficient of variation 0.15. The EC4 is conservative in predicting the strength of SRC columns with normal strength materials. The design code underestimates the resistance of SRC specimens. Thus, there is a need for EC4 to be modified for accurately predicting the strength of SRC columns.

[Table 3.3](#) Comparison between test and EC4

No. of Test	Mean	Standard deviation	COV

3.2.2 SRC columns with high strength materials

Although design values for SRC columns with normal strength materials are conservative as compared to experimental results, it does not mean the code is conservative for SRC columns with high strength materials. This is because high strength steel and concrete are not strain-compatible. The concrete reaches its peak strain much earlier than yield strain of high strength steel. This implies that concrete will fail earlier than steel and resulting in a partial utilization of capacities of both the materials. Thus, the applicability of EC4 for SRC columns with high strength materials exceeding their limits should be verified by test results. Table 3.4 shows the dimensions and material properties of SRC column specimens investigated by Kim et al. (2012&2014) with steel yield strength up to 913 MPa and concrete compressive strength up to 104 MPa.

Table 3.4 Dimensions and material properties of high strength SRC columns

Specimens	C1	C2	C3	C4	C10	C11
Section (mm)	260 x 260					
Steel section (mm)	150 x 100 x 17.6 x 17.6				150 x 150 x 15 x 15	
Longitudinal Rebar	6 – Φ 13	6 – Φ 13	6 – Φ 13	6 – Φ 13	6 – Φ 13	6 – Φ 13
f_{ck} (MPa)	91.3	91.3	91.3	91.3	104	104
f_y (MPa)	913	913	913	913	812	812
$f_{sk,l}$ (MPa)	525	525	525	525	512	512
Length L (mm)	2620	2620	2620	2620	2620	3520
Transverse rebar	Φ 10	Φ 10	Φ 10	Φ 10	Φ 10	Φ 10
$f_{sk,t}$ (Mpa)	560	560	560	703	474	474
Spacing (mm)	50	50	130	50	65	65
Eccentricity e_0 (mm)	120	60	120	120	120	120

The strengths of the 6 test specimens with high strength materials were predicted using the EC4 procedures mentioned in previous section. The comparison between

test and prediction value is shown in Table 3.5. It can be found that the average value of the resistance ratio ($N_{\text{test}}/N_{\text{EC4}}$) is 0.84. This observation indicates that the EC4 overestimates the ultimate strength of SRC columns because the full plastic method used in EC4 is not applicable for SRC columns with high strength materials. Strain compatibility problem should be addressed in the design of SRC columns and EC4 ought to be modified for predicting the strength of SRC columns with high strength steel and concrete to a good degree of accuracy.

Table 3.5 Comparison of test and predicted values using EC4

Specimen	N_{test} (kN)	M_{test} (kNm)	N_{EC4} (kN)	M_{EC4} (kNm)	$N_{\text{test}}/N_{\text{EC4}}$
C1	2203	336	2516	384	0.88
C2	3752	264	4242	299	0.88
C3	2020	270	2830	378	0.71
C4	2211	326	2594	382	0.85
C10	2329	366	2653	417	0.88
C11	2013	364	2345	424	0.86
Mean	-	-	-	-	0.84

3.3 Strain compatibility Method

For normal strength steel and concrete, the yield strain of steel is very close to the peak strain of concrete. The two materials are therefore strain-compatible and reach their full strengths when the composite concrete section reaches its ultimate strength. High strength steel and concrete are not strain-compatible in the sense that high strength concrete reaching its peak strain much earlier than yield strain of high strength steel. This implies that concrete will fail earlier than steel, resulting in a partial utilization of the capacities of both materials. The structural steel cannot develop its full strength, so the plastic method provided by EC4 is expected to overestimate the resistance of SRC columns with high strength steel. However, strain-compatibility method seems to be the most appropriate method for predicting the resistance of SRC columns. Using strain-compatibility method, the strength of

structural steel is limited to the stress corresponding to the crushing strain of concrete.

The yield strain of steel and the peak strain of concrete may be calculated in accordance with EC2 and EC3 as:

The yield strain of structural steel:

$$\varepsilon_y = f_y / E_a \quad (3.38)$$

The strain of concrete at peak stress (‰):

$$\varepsilon_c = 0.7 f_{cm}^{0.31} \leq 2.8 \quad (3.39)$$

where $f_{cm} = f_{ck} + 8$ is the mean compressive strength of concrete. The yield strain of steel and the strain of concrete at peak stress are respectively shown in [Table 3.6](#) and [Table 3.7](#). It is noted that the calculation for concrete strain ignores the confinement effect from the links and steel section.

To ensure the yield strain of steel less the compressive strain of concrete $\varepsilon_y \leq \varepsilon_c$, the maximum steel strength can be determined according to the following expression.

$$f_y \leq 0.7 E_a (f_{ck} + 8)^{0.31} \quad (3.40)$$

[Table 3.8](#) gives the maximum steel strength in SRC columns such that $\varepsilon_y < \varepsilon_{c1}$. Based on the recommendation for CFT from Liew (2015), a recommendation on the matching grades of steel and concrete suitable for use in SRC columns was proposed as shown in Table 3.9. It is recommended that the concrete class up to C90/105 may be used with steel grade up to S550 in SRC columns. The above calculation ignores the confinement effect from the links and steel section. If the increase of strain by confinement is taken into account, higher steel grade could be used in SRC columns.

Table 3.6 Yield strain of structural steel

Steel Grade	Yield strength f_y (N/mm ²)	Yield strain ϵ_y (‰)
S235	235	1.12
S275	275	1.31
S355	355	1.69
S420	420	2.00
S460	460	2.19
S500	500	2.38
S550	550	2.62
S620	620	2.95
S690	690	3.29

Table 3.7 Strain of concrete at peak stress

Concrete class	Cylinder strength f_{ck} (N/mm ²)	Mean value f_{cm} (N/mm ²)	Strain ϵ_{c1} (‰)
C20/25	20	28	1.97
C25/30	25	33	2.07
C30/37	30	38	2.16
C35/45	35	43	2.25
C40/50	40	48	2.32
C45/55	45	53	2.40
C50/60	50	58	2.46
C55/67	55	63	2.53
C60/75	60	68	2.59
C70/85	70	78	2.70
C80/95	80	88	2.80
C90/105	90	98	2.90

Table 3.8 Maximum strength of steel in SRC column

	S235	S275	S355	S420	S460	S500	S550	S620	S690
C20/25	235	275	355	413	413	413	413	413	413
C25/30	235	275	355	420	435	435	435	435	435
C30/37	235	275	355	420	454	454	454	454	454
C35/45	235	275	355	420	460	472	472	472	472
C40/50	235	275	355	420	460	488	488	488	488
C45/55	235	275	355	420	460	500	503	503	503
C50/60	235	275	355	420	460	500	518	518	518
C55/67	235	275	355	420	460	500	531	531	531
C60/75	235	275	355	420	460	500	544	544	544
C70/85	235	275	355	420	460	500	550	567	567
C80/95	235	275	355	420	460	500	550	589	589
C90/105	235	275	355	420	460	500	550	589	589

Table 3.9 Compatibility of steel and concrete materials for SRC column

	S235	S275	S355	S420	S460	S500	S550	S620	S690
C20/25	Yes	Yes	Yes	No	No	No	No	No	No
C25/30	Yes	Yes	Yes	Yes	No	No	No	No	No
C30/37	Yes	Yes	Yes	Yes	No	No	No	No	No
C35/45	Yes	Yes	Yes	Yes	Yes	No	No	No	No
C40/50	Yes	Yes	Yes	Yes	Yes	No	No	No	No
C45/55	Yes	Yes	Yes	Yes	Yes	Yes	No	No	No
C50/60	Yes	Yes	Yes	Yes	Yes	Yes	No	No	No
C55/67	Yes	Yes	Yes	Yes	Yes	Yes	No	No	No
C60/75	Yes	Yes	Yes	Yes	Yes	Yes	No	No	No
C70/85	Yes	Yes	Yes	Yes	Yes	Yes	Yes	No	No
C80/95	Yes	Yes	Yes	Yes	Yes	Yes	Yes	No	No
C90/105	Yes	Yes	Yes	Yes	Yes	Yes	Yes	No	No

Note: “Yes” indicates compatible materials and “No” is not recommended.

In the present study, the ultimate strain of test specimens is equal to the value obtained by Kim et al (2014). Thus the strength of steel is limited to 560 MPa. The resistance of test specimens was re-calculated using this modified limiting stress of steel, as shown in Table 3.10. N_{EC4} is the predicted value determined from the plastic method according to EC4 and $N_{EC4,u}$ is obtained from strain-compatibility method. These two methods are compared with test data. The mean value of the resistance ratio of plastic method and strain-compatibility method is 0.84 and 1.08 respectively. It indicates that the design value is conservative if EC4 approach with strain-compatibility considered is used to estimate the resistance of SRC columns with high strength material. Strain-compatibility method underestimates the test results because it neglects the lateral confinement effect from lateral hoops and steel section.

Table 3.10 Comparison of tests and code using stain compatibility method

Specimen	N_{test} (kN)	N_{EC4} (kN)	$N_{EC4,u}$ (kN)	N_{test}/N_{EC4}	$N_{test}/N_{EC4,u}$
C1	2203	2516	1901	0.88	1.16
C2	3752	4242	3414	0.88	1.10
C3	2020	2830	2127	0.71	0.95
C4	2211	2594	1957	0.85	1.13
C10	2329	2653	2124	0.88	1.09
C11	2013	2345	1900	0.86	1.06
Mean				0.84	1.08

3.4 Confined Concrete

The confining pressure from hoops and steel section can improve the ductility and strength of concrete. Analytical models were proposed by many researchers to investigate the stress-strain behaviour of confined concrete in RC members. Earlier studies showed that the behaviour of the confinement depended on various factors such as the shape of structural steel, the diameter, layout, spacing and number of longitudinal reinforcement and transverse rebar, and the yield strength of rebar as well as concrete strength. The SRC column was thus divided into highly confined, partially confined and unconfined concrete regions, as shown in Figure 3.4.

Figure 3.4 Confinement zones in SRC columns

However, analytical model for calculating the strength and strain of confined concrete of SRC columns is not addressed in current design codes. EC2 only provides the stress-strain relationship of confined concrete in RC members. The characteristic strength and strain of confined concrete is determined from:

$$\begin{aligned} f_{ck,c} &= f_{ck} (1.0 + 5.0 \sigma_2 / f_{ck}) & \text{for } \sigma_2 \leq 0.05 f_{ck} \\ f_{ck,c} &= f_{ck} (1.125 + 2.5 \sigma_2 / f_{ck}) & \text{for } \sigma_2 > 0.05 f_{ck} \end{aligned} \quad (3.41)$$

$$\varepsilon_{c2,c} = \varepsilon_{c2} \left(f_{ck,c} / f_{ck} \right)^2 \quad (3.42)$$

$$\varepsilon_{cu2,c} = \varepsilon_{cu2} + 0.2 \sigma_2 / f_{ck} \quad (3.43)$$

Where σ_2 is the lateral pressure due to confinement; ε_{c2} and $\varepsilon_{c2,c}$ are the strain of unconfined and confined concrete at peak stress respectively; ε_{cu2} and $\varepsilon_{cu2,c}$ are the ultimate strain of unconfined and confined concrete respectively.

Figure 3.5 Stress-strain curve of confined concrete (EC2)

The effective lateral compressive stress due to confinement can be determined according to EC8:

$$\sigma_2 / f_{ck} = 0.5 \alpha \omega_{wd} \quad (3.44)$$

Where:

ω_{wd} is the volumetric ratio of lateral hoops

$$\omega_{wd} = \frac{\text{Volume of hoops}}{\text{Volume of confined concrete}} \frac{f_{yd}}{f_{cd}}$$

α is the confinement factor, $\alpha = \alpha_n \alpha_s$, with:

$$\alpha_n = 1 - \frac{\sum b_i^2 / 6}{b_0 h_0} \quad (3.45)$$

$$\alpha_s = \left(1 - \frac{s}{2b_0}\right) \left(1 - \frac{s}{2h_0}\right) \quad (3.46)$$

Where n is the total number of longitudinal reinforcement laterally engaged by hoops; and b_i is the distance between consecutive engaged bars; h_0 is the depth of confined core; b_0 is the width of confined core; s is the spacing of hoops, as shown in Figure 3.6.

Figure 3.6 Dimension used in determining the confining effect

As the confinement model for SRC members is not provided in EC4, the confinement model for RC members is used in this comparison by neglecting the confinement from steel section. Therefore, the resistances of SRC column tests with high strength materials are calculated according to the confined concrete model in EC2. The comparison results are shown in Table 3.11.

Table 3.11 Comparison of test and Code considering confinement

Specimen	N_{test} (kN)	N_{EC4} (kN)	$N_{EC4,u}$ (kN)	$N_{EC4,c}$ (kN)	N_{test}/N_{EC4}	$N_{test}/N_{EC4,u}$	$N_{test}/N_{EC4,c}$
C1	2203	2516	1901	2351	0.88	1.16	0.94
C2	3752	4242	3414	4024	0.88	1.10	0.93
C3	2020	2830	2127	2224	0.71	0.95	0.91
C4	2211	2594	1957	2423	0.85	1.13	0.91
C10	2329	2653	2124	2421	0.88	1.09	0.96
C11	2013	2345	1900	2143	0.86	1.06	0.94
Mean					0.84	1.08	0.93
N_{EC4} is the resistance by EC4 plastic method; $N_{EC4,u}$ is the resistance by strain-compatibility method without considering confinement effect; $N_{EC4,c}$ is the resistance by strain-compatibility method considering confinement effect from hoops;							

It is shown that the average value of $N_{test}/N_{EC4,c}$ is 0.93 which means the value calculated by strain compatibility method with considering confinement effect from hoops overestimates the test results of SRC columns with high strength materials. Therefore, the confined concrete model given in EC2 cannot accurately predict the resistance of SRC columns with high strength materials, although EC2 model neglects the confinement effect from steel section. Thus, more accurate confinement models for confined concrete in SRC columns with high strength materials should be developed for design code EC4.

Chapter 4 Modified Confinement Model for SRC Column

The strength and ductility of concrete were enhanced by the confining pressure from lateral hoops and steel section based on earlier research works. As mentioned in Chapter 3, concrete in SRC columns can be divided into three regions: unconfined concrete region (UCC), partially confined concrete region (PCC) and highly confined concrete region (HCC).

For PCC, the confinement effect can be determined from the expressions for reinforced concrete columns which were extensively studied by Mander et al (1988), Cusson and Paultre (1995) and Legeron and Paultre (2003). Additionally, CEB Model code 1990, EC2 and FIB Model Code 2010 also provide expressions for calculating the stress-strain of confined concrete in RC columns. The confinement model provided in EC2 is not accurate for predicting the confinement effect which has been verified by tests data in previous section. Thus, an accurate confinement model should be identified from these models given in researcher papers or design codes.

For highly confined concrete, an analytical model was proposed by [Chen and Lin \(2005\)](#) to predict the behaviour of SRC columns. The ranges of two factors (partially confinement factor and highly confinement factor) are given in accordance with several experimental tests. The range of highly confinement factor is 1.1-1.97. The range of partially confinement factor is 1.09-1.5. However, the results are case-dependent, no interaction mechanism between concrete and steel section and formula is developed. Therefore, the study should focus on the effect of HCC on the performance of SRC members.

For these reasons, further study on the analytical model for SRC column should be carried out: 1) For PCC, confirming the most accurate confinement model from the published material models; 2) For HCC, developing the interaction mechanism between concrete and steel section. The modified confinement model for SRC columns can be developed by combining the verified PCC model and HCC model.

4.1 Material Models

4.1.1 Material Model for PCC

4.1.1.1 Mander Model

A stress-strain model for concrete confined by different shapes of transverse reinforcement was proposed by [Mander et al. \(1988\)](#). The stress-strain relationship shown in [Figure 4.1](#) can be determined from:

$$f_c = \frac{f'_{cc} x r}{r - 1 + x^r} \quad (4.1)$$

with

$$x = \frac{\varepsilon_c}{\varepsilon_{cc}} \quad (4.2)$$

$$r = \frac{E_c}{E_c - E_{\text{sec}}} \quad (4.3)$$

$$E_c = 5000 \sqrt{f'_{co}} \quad (4.4)$$

$$E_{\text{sec}} = \frac{f'_{cc}}{\varepsilon_{cc}} \quad (4.5)$$

$$f'_{cc} = f'_{co} \left(-1.254 + 2.254 \sqrt{1 + \frac{7.94 f'_l}{f'_{co}}} - 2 \frac{f'_l}{f'_{co}} \right) \quad (4.6)$$

$$\varepsilon_{cc} = \varepsilon_{co} \left[1 + 5 \left(\frac{f'_{cc}}{f'_{co}} - 1 \right) \right] \quad (4.7)$$

Where f'_{cc} and f'_{co} are the compressive strength of confined and unconfined concrete respectively; f'_l is the effective lateral confining stress; ε_{cc} and ε_{co} are the strain of confined and unconfined concrete respectively; E_c and E_{sec} are the tangent and secant modulus of confined concrete respectively.

The lateral confining pressure depends on strength and volume ratio of transverse reinforcement, layout of reinforcing steel and confined concrete area.

Figure 4.1 Stress-strain relationship from Mander model

As shown in Figure 4.2, the arching action occurs horizontally between longitudinal reinforcement and vertically between layers of transverse reinforcement. The confinement effective coefficient is calculated by the following:

$$k_e = \frac{\left(1 - \sum_{i=1}^n \frac{(w'_i)^2}{6b_c d_c}\right) \left(1 - \frac{s'}{2b_c}\right) \left(1 - \frac{s'}{2d_c}\right)}{1 - \rho_{cc}} \quad (4.8)$$

Where b_c and d_c are the width and depth of confined concrete respectively; s' is the spacing between lateral hoops; ρ_{cc} is the volumetric ratio of longitudinal reinforcement and w'_i is the distance between adjacent longitudinal reinforcement.

The ratio of lateral hoops to confined concrete in y and z direction is

$$\rho_y = \frac{A_{sy}}{s' d_c} \quad (4.9)$$

$$\rho_z = \frac{A_{sz}}{s' b_c} \quad (4.10)$$

where A_{sy} and A_{sz} are the lateral reinforcement area in corresponding directions.

The lateral confining pressure in y and z direction is

$$f_{ly} = \rho_y f_{yh} \quad (4.11)$$

$$f_{lz} = \rho_z f_{zh} \quad (4.12)$$

The effective lateral confining is

$$f'_l = k_e f_l \quad (4.13)$$

Figure 4.2 Effectively confined region

4.1.1.2 Legeron and Paultre Model

A confinement model for RC members was defined by [Cusson and Paultre \(1995\)](#). [Legeron and Paultre \(2003\)](#) modified this model based on strain compatibility, as shown in [Figure 4.3](#). The curve represents two critical points: (1) the maximum compressive strength f'_{cc} and strain ϵ'_{cc} ; (2) compressive strain ϵ_{cc50} corresponding to $0.5 f'_{cc}$.

The stress-strain relationship of confined concrete can be obtained from:

$$f_{cc} = \begin{cases} f'_{cc} \left[\frac{k(\varepsilon_{cc}/\varepsilon'_{cc})}{k-1+(\varepsilon_{cc}/\varepsilon'_{cc})^k} \right] & \varepsilon_{cc} \leq \varepsilon'_{cc} \\ f'_{cc} \exp \left[k_1(\varepsilon_{cc} - \varepsilon'_{cc})^{k_2} \right] & \varepsilon_{cc} \geq \varepsilon'_{cc} \end{cases} \quad (4.14)$$

With

$$k = \frac{E_{ct}}{E_{ct} - (f'_{cc}/\varepsilon'_{cc})} \quad (4.15)$$

$$k_1 = \frac{\ln 0.5}{(\varepsilon_{cc50} - \varepsilon'_{cc})^{k_2}} \quad (4.16)$$

$$k_2 = 1 + 25(I_{e50})^2 \quad (4.17)$$

Where f_{cc} and ε_{cc} are the stress and strain of confined concrete respectively; E_{ct} is tangent modulus of the unconfined concrete; I_{e50} is the effective confinement index.

Figure 4.3 Stress-strain curve from Legeron model

The maximum compressive stress and strain of confined concrete are:

$$\frac{f'_{cc}}{f'_c} = 1 + 2.4(I'_e)^{0.7} \quad (4.18)$$

$$\frac{\varepsilon'_{cc}}{\varepsilon'_c} = 1 + 35(I'_e)^{1.2} \quad (4.19)$$

With

$$I'_e = \frac{f'_{le}}{f'_c} \quad (4.20)$$

$$f'_{le} = \rho_{se} f'_h \quad (4.21)$$

$$\rho_{se} = k_e \rho_s \quad (4.22)$$

Where f'_c and ε'_c are the compressive strength and strain of unconfined concrete; k_e is the confinement coefficient introduced by [Mander et al \(1988\)](#); ρ_s is the volumetric ratio of hoops.

The stress of confining steel f'_h is given as:

$$f'_h = \begin{cases} f_h & \kappa \leq 10 \\ \frac{0.25 f'_c}{\rho_{se} (\kappa - 10)} \geq 0.43 \varepsilon'_c E_s & \kappa \geq 10 \end{cases} \quad (4.23)$$

With

$$\kappa = \frac{f'_c}{\rho_{se} E_s \varepsilon'_c} \quad (4.24)$$

The postpeak strain ε_{cc50} corresponding to 50% f'_{cc} is defined

$$\frac{\varepsilon_{cc50}}{\varepsilon_{c50}} = 1 + 60 I_{e50} \quad (4.25)$$

With

$$I_{e50} = \rho_{se} \frac{f_h}{f'_c} \quad (4.26)$$

Where ε_{c50} is the postpeak strain of unconfined concrete measured at $0.5 f'_c$.

4.1.1.3 Fib Model Code 2010

The Fib (federation internationale du beton) Model Code for Concrete Structures 2010 is an international code. The earlier version CEB Model Code 1990 is served as an important basis for EC2.

Fib model code 2010 defines the stress-strain relationship as following

$$f_c = (f_{ck,c} + 8) \left(\frac{k\eta - \eta^2}{1 + (k-2)\eta} \right) \quad (4.27)$$

With

$$\eta = \frac{\varepsilon_c}{\varepsilon_{c,c}} \quad (4.28)$$

$$k = \frac{E_{cm}}{E_{c1}} \quad (4.29)$$

Where E_{cm} and E_{c1} are tangent and secant modulus of concrete respectively.

The characteristic strength and strain of confined concrete can be determined from:

$$\frac{f_{ck,c}}{f_{ck}} = 1.0 + 3.5 \left(\frac{\sigma_2}{f_{ck}} \right)^{0.75} \quad (4.30)$$

$$\varepsilon_{c,c} = \varepsilon_c \left[1 + 5 \left(\frac{f_{ck,c}}{f_{ck}} - 1 \right) \right] \quad (4.31)$$

$$\varepsilon_{cu,c} = \varepsilon_{cu} + 0.2 \sigma_2 / f_{ck} \quad (4.32)$$

The effective lateral compressive stress can be determined by

$$\sigma_2 = \alpha \omega_c f_{cd} \quad (4.33)$$

with

$$\omega_c = \min \left\{ \omega_y = \frac{A_{sy} f_{yd}}{b_c s f_{cd}}, \omega_z = \frac{A_{sz} f_{yd}}{h_c s f_{cd}} \right\} \quad (4.34)$$

$$\alpha = \left(1 - \frac{\sum b_i^2 / 6}{b_c h_c} \right) \left(1 - \frac{s}{b_c} \right) \left(1 - \frac{s}{h_c} \right) \quad (4.35)$$

Where h_c and b_c are the depth and width of confined core respectively.

4.1.2 Material Model for HCC

In a SRC column, the steel section can provide lateral confining pressure through the contribution of bending stiffness of the steel flanges. Mander model, Legeron model, Fib model or EC2 Model which is proposed for reinforced concrete, are applicable for HCC in SRC column once the lateral confining stress from steel section is clear. In SRC column, HCC is confined by steel section and the hoops together. Hence, the effective confining stress for HCC is obtained from:

$$f'_{lh} = f'_{lp} + f'_{ls} \quad (4.36)$$

where f'_{lp} and f'_{ls} are the effective lateral confining stress from hoops and steel section respectively.

4.1.2.1 Lateral confining stress from steel section

Concrete between the flanges of structural steel section in SRC column is highly confined than concrete elsewhere. The effective confined zone is considered to be the total concrete area minus the area of the two parabolas with height one quarter their length, as shown in [Figure 4.4](#). [EI-Tawil \(1996\)](#) suggested that the confinement effectiveness can be obtained from the following expression:

$$k'_e = \frac{\text{total concrete area between flanges-area of parabolas}}{\text{total concrete area between flanges}} \quad (4.37)$$

Figure 4.4 Highly confined region between steel flanges

The confining pressure can be evaluated based on the resistance from the flanges. The moment applied at the base of steel flange due to the expanding concrete should be equal to the resistance developed by the flanges, as shown in Figure 4.5. So

$$\frac{f'_{ly}}{k'_e} \frac{l^2}{2} = \frac{t^2 f_y}{6} \quad (4.38)$$

The lateral confining pressure is given by

$$f'_{ly} = k'_e \frac{t^2 f_y}{3l^2} \quad (4.39)$$

where f'_{ly} is the additional confining pressure due to steel section; t is the thickness of steel flange; l is length of steel flange measured from face of web.

Figure 4.5 Confining pressure due to flanges

4.2 Modified Confinement Model for SRC Columns

Modified confinement models for SRC columns were defined by combining the confinement model for PCC and confinement effect from steel section. The modified confinement model 1 was proposed by combining the Legeron model for PCC and EI-Tawil model for HCC. According to the modified confinement model, the strength and strain of PCC and HCC are expressed as follows:

$$\left\{ \begin{array}{l} f_{c,p} = K_{p,l} f_{c,u} = \left(1 + 2.4 \left(\frac{f_{l,p}}{f_{c,u}} \right)^{0.7} \right) f_{c,u} \\ \varepsilon_{c,p} = K_{p,l} \varepsilon_c = \left(1 + 35 \left(\frac{f_{l,p}}{f_{c,u}} \right)^{1.2} \right) \varepsilon_c \end{array} \right. \quad (4.40)$$

$$\left\{ \begin{array}{l} f_{c,h} = K_{h,I} f_{c,u} = \left(1 + 2.4 \left(\frac{f_{l,h}}{f_{c,u}} \right)^{0.7} \right) f_{c,u} \\ \varepsilon_{c,h} = K_{h,II} \varepsilon_c = \left(1 + 35 \left(\frac{f_{l,h}}{f_{c,u}} \right)^{1.2} \right) \varepsilon_c \end{array} \right. \quad (4.41)$$

Where $K_{p,I}$, $K_{p,II}$, $K_{h,I}$ and $K_{h,II}$ are confinement factors for PCC and HCC respectively; $f_{l,p}$ and $f_{l,h}$ are the lateral confining stress for PCC and HCC respectively; $f_{c,u}$ and ε_c are the strength and strain of unconfined concrete.

1) Effective lateral confining pressure on PCC

The effective lateral confining pressure for PCC from hoops can be determined by the follow:

$$f_{l,p} = k_e \rho_{sh} f_{r,h} \quad (4.42)$$

where

ρ_{sh} is the effective volume ratio of hoops;

k_e is the area ratio of effectively confined area in PCC

$$k_e = \frac{\left(1 - \sum_{i=1}^n \frac{(b_i)^2}{6b_c h_c} \right) \left(1 - \frac{s}{2b_c} \right) \left(1 - \frac{s}{2h_c} \right)}{1 - \rho_s} \quad (4.43)$$

$f_{r,h}$ is the real stress of hoops. Experimental studies conducted by [Sheikh and Uzumeri \(1980\)](#) showed that high strength hoops may not yield when the confinement effect is insufficient. Thus, the real stress of hoops needs to be determined and the procedure proposed by [Legeron and Paultre \(2003\)](#) was adopted.

$$f_{r,h} = \begin{cases} f_{s,h} & \kappa \leq 10 \\ \frac{0.25 f_{c,u}}{k_e \rho_{sh} (\kappa - 10)} \geq 0.43 \varepsilon_c E_s & \kappa \geq 10 \end{cases} \quad (4.44)$$

where

$$\kappa = \frac{f_{c,u}}{k_e \rho_{sh} E_s \varepsilon_c} \quad (4.45)$$

2) Effective lateral confining stress on HCC

The effective lateral confining stress for HCC can be determined by the follow:

$$f_{l,h} = f_{l,p} + f_{l,s} \quad (4.46)$$

Where

$f_{l,s}$ is the effective lateral confining stress from steel section;

$$f_{l,s} = k'_e k_a f_{r,y} \quad (4.47)$$

k'_e is the confinement effectiveness;

$$k'_e = \frac{A_{c,f} - A_{c,r}}{A_{c,f}} \quad (4.48)$$

$A_{c,f}$ is the total concrete area between steel flange;

$A_{c,r}$ is the area of the two parabolas with height one quarter their length;

k_a is confining facor;

$$k_a = \frac{t_f^2}{3l^2} \quad (4.49)$$

t_f is the steel flange thickness;

l is length of steel flange measured from face of web;

$f_{r,y}$ is the real stress of steel section.

The other three modified confinement models 2, 3 and 4 were defined by combining the corresponding confinement model (Mander, EC2, FIB) for PCC and EI-Tawil for confinement effect from steel section.

4.3 Analytical Model for SRC columns

The resistance of SRC columns under axial loading was determined by full plastic method in accordance with EC4. Based on earlier discussion, the full plastic method overestimated the resistance of SRC columns with high strength steel. Strain-compatibility issue should be considered in the design. Therefore, an analytical model was proposed to estimate the SRC column strength by considering the confinement effect from lateral hoops and steel section and the strain-compatibility method. The analytical axial resistance N_p is expressed as follows:

$$N_p = A_a f_{r,y} + 0.85(A_{ch} f_{c,h} + A_{cp} f_{c,p} + A_{cu} f_{c,u}) + A_s f_{r,s} \quad (4.50)$$

Where A_{cu} , A_{cp} and A_{ch} are area of UCC, PCC and HCC respectively; $f_{r,y}$ and $f_{r,s}$ are the real stress of steel section and reinforcement considering strain-compatibility; $f_{c,u}$, $f_{c,p}$ and $f_{c,h}$ are the compressive strength of UCC, PCC and HCC respectively.

For design purpose, the stress of individual material considering the partial factor was used in equation (4.50).

4.4 Comparison of Material Models

The accuracy of the four modified confinement models was verified by SRC column tests by Kim et al. (2012&2014). The comparison of these modified models is given in Table 4.1 and Figure 4.6. The compressive strength ratio of PCC to UCC ($f_{c,p}/f_{c,u}$) varies from 1.02 to 1.36, and the ratio $f_{c,h}/f_{c,u}$ varies from 1.29 to 2.47, as shown in Figure 4.6(a) and (c). On the other hand, the ratio of PCC strain to UCC strain varies from 1.02 to 2.83, and the ratio of HCC strain to UCC strain varies from 1.73 to 8.42 as shown in Figure 4.6 (b) and (d).

For partially confined concrete, the values calculated from the EC2 model and Fib model are higher than the other two models, while for highly confined concrete, the values predicted by modified Mander model and Fib model are higher than the other two models. It can be seen that the modified Legeron model is the most conservative one for predicting the stress-strain of confined concrete. Additionally, it is shown that the spacing of links has a significant effect on the stress-strain of

confined concrete. For example, the strength and strain of specimen C3 is lower than other specimens because the spacing of links is 130mm for test specimen C3, while it is 50 mm for C1, C2.

Additionally, the comparison of column strengths from test and analytical model are given in Table 4.2. It is shown that the predicted values from analytical model based on modified EC2 model, Mander model and Fib model are generally the same. The mean value of the analytical to test results ratio are 1.26, 1.29 and 1.21 respectively. Thus these three material models are unconservative. However, the modified Legeron Model is more conservative as compared to the three models. The mean value is 1.03 which means that this model can accurately predict the SRC columns strength.

Therefore, the load-strain curves of specimens were obtained from analytical model with consideration of the modified confinement model from Legeron and EI-Tawil for PCC and HCC in SRC columns. The comparison of load-strain curves between analytical model and experiment investigation is shown in Figure 4.7. It was observed that the analytical curve agreed well with the experimental value. Therefore, the modified confinement model can be used in the numerical simulation and design in the forthcoming sections. The analytical model can be used to predict the behaviour of SRC columns.

(a) Strength ratio of PCC and UCC ($f_{c,p}/f_{c,u}$)

(b) Strain ratio of PCC and UCC ($\varepsilon_{c,p}/\varepsilon_c$)

Strength ratio of HCC and UCC ($f_{c,h}/f_{c,u}$)

Figure 4.6 Comparison of material models

Table 4.1 Comparison of Material models

Test	C1	C2	C3	C4	C10	C11
$f_{c,u}$ (Mpa)	94	94	94	94	104	104
ε_c (‰)	2.649	2.649	2.649	2.649	2.6	2.6
$f_{c,p,EC2}$ (MPa)	121	121	104	126	123	123
$\varepsilon_{c,p,EC2}$ (‰)	4.4	4.4	3.2	4.7	3.8	3.8
$f_{c,h,EC2}$ (MPa)	194	194	159	180	140	140
$\varepsilon_{c,h,EC2}$ (‰)	11.1	11.1	7.5	9.6	4.8	4.8
$f_{c,p,Mander}$ (MPa)	118	118	101	122	118	118
$\varepsilon_{c,p,Mander}$ (‰)	5.9	5.9	3.6	6.5	4.5	4.5
$f_{c,h,Mander}$ (MPa)	229	229	207	231	168	168

$\varepsilon_{c,h,Mander} (\%)$	21.6	21.6	18.4	21.9	11.0	11.0
$f_{c,p, Fib} \text{ (MPa)}$	122	122	102	128	123	123
$\varepsilon_{c,p,Fib} (\%)$	6.6	6.6	3.8	7.5	5.2	5.2
$f_{c,h, Fib} \text{ (MPa)}$	244	244	216	232	164	164
$\varepsilon_{c,h,Fib} (\%)$	23.7	23.7	19.7	22.1	10.5	10.5
$f_{c,p, Legeron} \text{ (MPa)}$	106	106	98	106	112	112
$\varepsilon_{c,p,Legeron} (\%)$	3.2	3.2	2.7	3.2	3.0	3.0
$f_{c,h, Legeron} \text{ (MPa)}$	178	178	166	177	134	134
$\varepsilon_{c,h,Legeron} (\%)$	14.6	14.6	11.9	14.6	4.5	4.5

Table 4.2 Comparison of analytical models and test results

Test	N_{Test} (kN)	N_{EC2} (kN)	N_{Mander} (kN)	N_{Fib} (kN)	$N_{Legeron}$ (kN)	$\frac{N_{EC2}}{N_{test}}$	$\frac{N_{Mander}}{N_{test}}$	$\frac{N_{Fib}}{N_{test}}$	$\frac{N_{Legeron}}{N_{test}}$
C1	2203	2729	2745	2461	2166	1.24	1.24	1.12	0.98
C2	3752	5186	5213	4698	3862	1.38	1.39	1.25	1.03
C3	2020	2474	2666	2435	2236	1.22	1.32	1.20	1.10
C4	2211	2836	2852	2564	2271	1.28	1.29	1.16	1.03
C10	2329	2743	2793	2815	2341	1.18	1.20	1.21	1.0
C11	2013	2563	2639	2629	2076	1.27	1.31	1.30	1.03
Mean	-	-	-	-	-	1.26	1.29	1.21	1.03

(a) C1

(b) C2

(c) C10

(d) C11

Figure 4.7 Comparison of analytical and Test curves

Chapter 5 Finite Element Modelling

5.1 Finite Element Modelling

5.1.1 Modelling Approach

In this study, the finite element package ABAQUS (2014) was used to analyse the behaviour of axially loaded SRC column. The concrete (highly confined, partially confined and unconfined concrete), steel section, reinforcement (longitudinal and transverse) and the interface of steel section-concrete and reinforcement-concrete have been modelled using the approach proposed by [Ellobody & Young \(2011\)](#). [Figure 5.1](#) shows the FE model of SRC column. In this study, the highly confined concrete area was considered to be half of the concrete area between the two flanges of steel section. The concrete cover region was considered as the unconfined concrete. The remaining area was defined as the partially confined concrete region.

(d) Highly confined concrete   (e) Partially confined concrete   (f) End plate

Figure 5.1 FE model of SRC columns

5.1.2 Material Definition

5.1.2.1 Concrete

The damaged plasticity model was used to model the behaviour of concrete. The definition of this model consisted of three parts: plasticity, compressive behaviour and tensile behaviour. The parameters of the plasticity were adopted as following: the dilation angle was 30° ; eccentricity was 0.1; the initial equibiaxial to uniaxial compressive yield stress ratio (f_{b0}/f_{c0}) was 1.16 and k was 0.667.

The stress-strain relationship of PCC and HCC was defined by the modified confinement model in previous section, depending on Legeron model (2003) and EI-Twail Model (1996), as shown in Figure 5.2(a). The concrete area in SRC column is divided into three parts: UCC region (concrete cover), PCC region (confined by hoops), and HCC region (confined by hoops and steel section). In Figure 5.2(a), $\varepsilon_{c,u}$, $\varepsilon_{c,p}$ and $\varepsilon_{c,h}$ are the strain of UCC, PCC and HCC at peak stress respectively, $\varepsilon_{c50,u}$, $\varepsilon_{c50,p}$ and $\varepsilon_{c50,h}$ are the ultimate compressive strain corresponding to half of the peak stress of UCC, PCC and HCC respectively. The tensile strength of concrete model proposed by Ellobody & Young (2011) was used

in this model. Figure 5.2 (b) represents the response of concrete to uniaxial tension. Additionally, the elastic modulus was defined in accordance with EC2; the Poisson's ratio 0.2 was considered.

(a) Compression

(b) Tension

Figure 5.2 Stress-strain curve of confined concrete

5.1.2.2 Structural Steel and Reinforcement

The stress-strain relationships of steel section and reinforcement were defined in accordance with EC3 and EC2 respectively. It was assumed that the stress-strain

responses in compression and tension were same. In ABAQUS, the Young's modulus and Poisson's ratio of 210 GPa and 0.3 were considered.

5.1.3 Modelling of Interfaces

The contact interaction between steel and concrete was defined as surface to surface contact. Two surfaces, master surface and slave surface, were defined according to ABAQUS. The concrete and steel in the FE model were defined as the master surface and slave surface respectively. A condition of non penetration was assumed in the FE analysis. The coefficient of friction at the interface was defined as 0.25 according to FE model by [Ellobody and Young \(2011\)](#).

5.1.4 Finite Element Type and Mesh

3-D solid elements were used to model the SRC column in this study. The elements had three degrees of freedom per node. The effect of various mesh sizes in SRC column was investigated to choose a reasonable size that provided accurate results and minimized computational effort. A FE mesh size of 10 mm was considered in this study.

5.1.5 Loading Definition and Boundary Conditions

Two rigid plates (20mm thickness) were modelled and connected to both ends of SRC column in the study. Both ends were hinged in the numerical simulation. The load was applied to the rigid plates according to the experimental investigation of test specimens. The loading point was allowed to be displaced in the vertical direction (along the column length).

5.2 Verification of Finite Element Model

5.2.1 Test Results

The finite element model of SRC columns was verified against tests data from Matsui (1979), SSRC Task Group 20 (1979), Morino (1984) and Kim et al (2012&2014). [Table 5.1](#) shows the dimensions and material properties of test specimens.

The test conducted by Matsui (1979) were performed on SRC columns with reinforcing steel, as shown in [Figure 5.3 \(a\)](#). The dimension of the cross-section was

160 mm $\times$ 160 mm and the column length was 924 mm, 2309 mm and 3464 mm respectively. The dimension of steel section was 100 $\times$ 100 $\times$ 6 $\times$ 8 mm. The diameter of longitudinal and transverse reinforcement was 6 mm and 4 mm respectively. The spacing of transverse reinforcement was 75 mm. The concrete strength of specimens varied from 18.5 to 22.5 MPa. The range of yield strength of structural steel was 298-306 MPa. The yield strength of reinforcement is 376 MPa.

The dimension of the specimen conducted by SSRC Task Group 20 (1979) was 165.1 mm $\times$ 177.8 mm and the column length was 1168 mm and 1156 mm respectively. The dimension of steel section was UB 127 $\times$ 114 $\times$ 29.76. The longitudinal and transverse reinforcement were same as Matsui (1979). The concrete strength was 18 and 19.3 MPa, and the yield strength of steel was 248 and 232 MPa. The yield strength of reinforcement was 376 and 460 MPa respectively.

The cross-sectional dimension and steel section of test specimens conducted by Morino (1984) were similar to test specimens by Matsui (1979). The concrete strength was 21.1 and 23.4 MPa. The yield strength of steel section and reinforcement was 345 and 460 MPa respectively.

The details of SRC column tests conducted by Kim et al (2012&2014) are shown in [Figure 5.3 \(b\)](#). The dimension of cross-section was 260 $\times$ 260 mm and column lengths were 2620 mm and 3520 mm. The dimensions of steel section were 150 $\times$ 100 $\times$ 17.6 $\times$ 17.6 mm and 150 $\times$ 150 $\times$ 15 $\times$ 15 mm respectively. The diameter of longitudinal reinforcement and transverse reinforcement was 13 mm and 10 mm respectively. The yield strength of steel was 812 MPa and 913 MPa. The compressive strength of concrete was 94 MPa and 104 MPa. The yield strength of longitudinal reinforcement was 512 and 525 MPa respectively, and the yield strength of lateral hoops was 474, 560 and 703 MPa respectively. The effect of the spacing of hoops on the behaviour of SRC columns was also investigated in these tests. The spacing of the hoops was 50 mm, 65 mm and 130 mm respectively, as shown in [Figure 5.4](#).

Figure 5.3 Details of SRC columns test

Table 5.1(a) Dimensions and material properties of SRC column test

Test		Dimension			Structural steel section (mm)	Eccentricity (mm)
		b_c (mm)	h_c (mm)	L (mm)		
G1	A1	160	160	924	$100 \times 100 \times 6 \times 8$	-
	A2	160	160	2309	$100 \times 100 \times 6 \times 8$	-
	A3	160	160	3464	$100 \times 100 \times 6 \times 8$	-
G2	B1	165.1	177.8	1168	UB127 $\times$ 114 $\times$ 29.26	-
	B2	165.1	177.8	1156	UB127 $\times$ 114 $\times$ 29.26	12.5
G3	C1	260	260	2620	$150 \times 100 \times 17.6 \times 17.6$	120
	C2	260	260	2620	$150 \times 100 \times 17.6 \times 17.6$	60
	C3	260	260	2620	$150 \times 100 \times 17.6 \times 17.6$	120
	C4	260	260	2620	$150 \times 100 \times 17.6 \times 17.6$	120
	C10	260	260	2620	$150 \times 150 \times 15 \times 15$	120
	C11	260	260	3520	$150 \times 150 \times 15 \times 15$	120
G4	D1	160	160	960	$100 \times 100 \times 6 \times 8$	40
	D2	160	160	2400	$100 \times 100 \times 6 \times 8$	40

Table 5.1(b) Dimensions and material properties of SRC column test

Test		Reinforcement		Material			
		Longitudinal	Transverse	f_c (MPa)	f_a (MPa)	$f_{s,l}$ (MPa)	$f_{s,h}$ (MPa)
G1	A1	4 $\Phi 6$	$\Phi 4$ @75mm	18.5	306	376	376
	A2	4 $\Phi 6$	$\Phi 4$ @75mm	21.4	298	376	376
	A3	4 $\Phi 6$	$\Phi 4$ @75mm	22.5	304	376	376
G2	B1	4 $\Phi 6$	$\Phi 4$ @75mm	18	248	376	376
	B2	4 $\Phi 6$	$\Phi 4$ @75mm	19.3	232	460	460
G3	C1	6 $\Phi 13$	$\Phi 10$ @50 mm	94	913	525	560
	C2	6 $\Phi 13$	$\Phi 10$ @50 mm	94	913	525	560
	C3	6 $\Phi 13$	$\Phi 10$ @130 mm	94	913	525	560
	C4	6 $\Phi 13$	$\Phi 10$ @50 mm	94	913	525	703
	C10	6 $\Phi 13$	$\Phi 10$ @65 mm	104	812	512	474
	C11	6 $\Phi 13$	$\Phi 10$ @65 mm	104	812	512	474
G4	D1	4 $\Phi 6$	$\Phi 4$ @75mm	21.1	345	460	460
	D2	4 $\Phi 6$	$\Phi 4$ @75mm	23.4	345	460	460

Figure 5.4 Reinforcement layout with spacing 50, 65 and 130 mm

5.2.2 Comparison between FE and Test Results

The ultimate strength obtained from experimental investigation (N_{test}) and finite element analysis (N_{FE}) was compared in [Table 5.2](#). It was observed that the finite element result agrees well with test value. The maximum difference between test data and FE result is 9 %. The mean value of $\frac{N_{\text{FE}}}{N_{\text{Test}}}$ is 1.0 and the COV is 0.057.

The predicted value calculated by the EC4 approach (N_{EC4}) and the proposed method (N_{p}) were also compared with the test data. The average value of $\frac{N_{\text{Test}}}{N_{\text{EC4}}}$ is

1.05 with COV of 0.209, and the mean value of $\frac{N_{\text{Test}}}{N_{\text{p}}}$ is 1.09 with COV of 0.124.

Although the mean value of $\frac{N_{\text{Test}}}{N_{\text{EC4}}}$ is less than the $\frac{N_{\text{Test}}}{N_{\text{p}}}$, this does not mean that the

EC4 approach is more accurate because the COV is larger than the proposed method. The test specimens were divided into two groups. For group G1, G2 and G4, normal strength materials were used in SRC column test. For these three groups, the mean value of $\frac{N_{\text{FE}}}{N_{\text{Test}}}$ is 1.23 with COV of 0.112, and the mean value of $\frac{N_{\text{Test}}}{N_{\text{p}}}$ is

1.19 with COV of 0.114. For group G3, high strength materials were used in SRC column tests. The mean value of $\frac{N_{\text{FE}}}{N_{\text{Test}}}$ is 0.84 with COV of 0.056, and the mean

value of $\frac{N_{\text{Test}}}{N_{\text{p}}}$ is 0.97 with COV of 0.029. It was observed that the strength

calculated from the proposed method (N_{p}) was more accurate than EC4 method (N_{EC4}), as shown in [Figure 5.5](#).

Table 5.2 Comparison between test data and FE result

Test		N_{test} (kN)	N_{FE} (kN)	N_{EC4} (kN)	N_{P} (kN)	$\frac{N_{\text{FE}}}{N_{\text{Test}}}$	$\frac{N_{\text{Test}}}{N_{\text{EC4}}}$	$\frac{N_{\text{Test}}}{N_{\text{P}}}$
G1	A1	996	1004	867	896	1.01	1.15	1.11
	A2	974	905	753	774	0.93	1.29	1.26
	A3	874	817	615	628	0.94	1.42	1.39
G2	B1	1370	1300	1125	1179	0.95	1.22	1.16
	B2	996	1078	909	944	1.08	1.10	1.05
G3	C1	2203	2336	2516	2166	1.06	0.88	1.02
	C2	3752	3397	4242	3862	0.91	0.88	0.97
	C3	2020	2175	2830	2236	1.08	0.71	0.90
	C4	2211	2221	2594	2271	1.0	0.85	0.97
	C10	2329	2224	2653	2341	0.95	0.88	0.99
	C11	2013	2076	2345	2076	1.03	0.86	0.97
G4	D1	740	711	559	587	0.96	1.32	1.26
	D2	504	535	449	464	1.06	1.12	1.09
Mean		-				1.0	1.05	1.09
COV		-				0.057	0.209	0.124

Figure 5.5 Comparison of EC4 and Proposed Methods

There are three failure modes for SRC columns under concentric axial loading based on the test and FE results: concrete crushing, steel yielding and flexural buckling. The three modes are easily identified by comparing the stresses of concrete elements and steel elements with material strengths. Generally, for SRC column with normal strength material, concrete crushing and steel yielding occur simultaneously if the relative slenderness ratio of SRC column is low enough. In this failure mode, the concrete fails in compression and the steel reaches its yield stress simultaneously. [Figure 5.6](#) shows a typical stress distribution and deformed shape of SRC column. The failure mode combined the concrete crushing and steel yielding. It is shown that the compressive stress was concentrated near the end of SRC columns.

However, if the relative slenderness ratio of SRC column exceeds a certain value, the flexural buckling failure mode occurs. In this type of failure mode, the full strength of steel and concrete are not reached. [Figure 5.7](#) shows a typical deformed shape of flexural buckling mode. For SRC column with high strength material (e.g. C1-C4 & C10-C11), the specimen failed due to early crushing of concrete, without yielding of steel. The failure mode of specimen C1 under experimental investigation is shown in [Figure 5.8](#). Similarly, in finite element analysis, the stress of steel at failure was less than the yield strength, which verified the test results.

[Figure 5.6](#) Deformation and stress distribution of short column

Figure 5.7 Deformation of slender column

Figure 5.8 Failure mode of specimen C1 and steel stress in FE simulation

Chapter 6 Parametric Study and Discussion

6.1 SRC Columns under Concentric Axial Loading

6.1.1 Parameters

The verified FE model was used in the parametric study. Parameters including concrete strength, yield strength of structural steel and reinforcement, column slenderness, spacing, diameter and volumetric ratio of transverse reinforcement were considered. A total of 216 axially loaded SRC columns were analysed in the parametric study. The dimensions and material properties of SRC columns are shown in [Table B1 of Appendix B](#).

In the study, the cylinder strength of concrete f_c was varied from normal strength 50 Mpa to high strength 90 MPa. The yield strength of structural steel f_y was varied from 355 MPa to 690 MPa. The yield strength of reinforcement f_s was 500 Mpa. The spacing of transverse reinforcement S was from 50 mm to 150 mm, and the diameter of transverse reinforcement Φ was 8 mm and 6 mm respectively. The spacing and diameter of transverse reinforcement was replaced by one parameter volumetric ratio ρ_{sh} . The volumetric ratio of transverse reinforcement was varied from 0.5% to 2.8%. The value of SRC column lengths L were 800 mm, 1600 mm and 2400 mm respectively. As the columns were pin-ended, the effective buckling length was the overall column length. The relative slenderness ratio was varied from 0.2 to 0.81 which was calculated from the expression given in EC4. The values of parameters are given in [Table 6.1](#).

[Table 6.1](#) Values of parameters

L (mm)	f_c (MPa)	f_y (MPa)	S (mm)	Φ (mm)	ρ_{sh} (%)
800	50	355	50	6	0.5
1600	70	460	100	8	0.8
2400	90	550	150		0.9
		690			1.4

					1.5
					2.8

6.1.2 Effect of Parameters on Column strength

6.1.2.1 Effect of Effective Column Length

Figure 6.1 shows the effect of effective length and concrete strength on the ultimate strength of SRC columns. For short column, the ultimate strength of SRC column increases significantly with the increment of concrete compressive strength. However, for slender column, the SRC column strength increases marginally by increasing concrete strength. It means that the increment in concrete strength has a negligible effect on the bearing capacity of SRC columns with larger slenderness ratio, as flexural buckling failure governs the damage mode. Therefore, the usage of high strength concrete is only beneficial for short columns. The ultimate strength of SRC column is improved by 45% when the concrete strength increases from 50 Mpa to 90 Mpa.

(c) S550, $\Phi = 8$ mm, $S = 50$ mm

(d) S690, $\Phi = 8$ mm, $S = 50$ mm

(e) S355, $\Phi = 8$ mm, $S = 100$ mm

(f) S460, $\Phi = 8$ mm, $S = 100$ mm

(g) S550, $\Phi = 8$ mm, $S = 100$ mm

(h) S690, $\Phi = 8$ mm, $S = 100$ mm

(i) S355, $\Phi = 8$ mm, $S = 150$ mm

(j) S460, $\Phi = 8$ mm, $S = 150$ mm

(k) S550, $\Phi = 8$ mm, S = 150 mm

(l) S690, $\Phi = 8$ mm, S = 150 mm

Figure 6.1 Effect of length and concrete strength on strength of SRC column

Figure 6.2 shows the effect of column length and steel strength on the strength of SRC columns. Similar to the previous case, for short column, the ultimate load of SRC column increases significantly when the steel strength increases. However, for slender column, the ultimate load increases marginally with the increase in steel strength. Again, the effect of steel strength on the ultimate strength of SRC column having large slenderness ratio is negligible due to flexural buckling failure mode. Thus, the use of high strength steel in short columns would only result in a significant improvement in the ultimate strength of SRC column. The axial load bearing capacity of SRC column is improved by 25% when steel strength increases from 355 to 690 Mpa. The effect of increase in steel strength is lower due to a lower strength contribution ratio from steel.

(a) C50, $\Phi = 8$ mm, S = 50 mm

(b) C70, $\Phi = 8$ mm, S = 50 mm

(c) C90, $\Phi = 8$ mm, $S = 50$ mm

(d) C50, $\Phi = 8$ mm, $S = 100$ mm

(e) C70, $\Phi = 8$ mm, $S = 100$ mm

(f) C90, $\Phi = 8$ mm, $S = 100$ mm

(g) C50, $\Phi = 8$ mm, $S = 150$ mm

(h) C70, $\Phi = 8$ mm, $S = 150$ mm

(i) C90, $\Phi = 8$ mm, S = 150 mm

Figure 6.2 Effect of column length and steel strength on strength of SRC column

6.1.2.2 Effect of Material Strength

Figure 6.3 represents the effect of material strength on the bearing capacity of SRC column. It can be seen that the strength of SRC column increases linearly with increasing in concrete strength. The ultimate strength of column increases linearly with increasing in steel strength when the spacing of hoops is equal to 50 mm, as shown in Figure 6.3 (a). For steel strength $f_y \leq 550$ Mpa, the ultimate strength of column increases linearly by increasing steel strength. However, for steel strength $f_y > 550$ Mpa, increase in the steel strength has a smaller influence on the ultimate strength for columns having larger spacing of hoops due to insufficient confinement, as shown in Figure 6.3 (b)-(c). The yield strain of steel grade S690 is greater than the crushing strain of concrete due to insufficient confinement. Therefore, the full strength of high strength steel cannot be developed. The ultimate strength of column is limited by early concrete crushing.

(a) Spacing of links (50 mm)

(b) Spacing of links (100mm)

(c) Spacing of links (150mm)

Figure 6.3 Effect of material strength on strength of SRC column ($l = 800$ mm)

6.1.2.3 Effect of Hoops

In the parametric study, the diameter of transverse reinforcement (Hoops) was 6mm and 8 mm respectively. The spacing of hoops was 50 mm, 100 mm, and 150 mm respectively. Therefore, the volumetric ratio of hoops to confined concrete was varied from 0.5% to 2.8%. The effect of hoops on the bearing capacity of SRC column is shown in Figure 6.4 – 6.6.

The effect of spacing of hoops on the column strength is represented in Figure 6.4. It shows that the bearing capacity of column increases when the hoops spacing decreases. For steel grade S355, S460 and S550, the improvement of column strength is less than that of steel grade S690. The reason is that concrete crushing and steel yielding occur when the column reaches its ultimate strength for steel strength $f_y \leq 550$ Mpa. The ultimate strength of column increases by increasing the confined concrete strength. However, for steel strength $f_y > 550$ Mpa, early concrete crushing occurs prior to steel yielding. With the spacing of hoops decreasing, the lateral confinement level becomes higher. The full strength of steel may be developed due to the increased concrete strain. In this case, the strength of column increases due to the increased strength of confined concrete and steel.

Figure 6.4 Effect of spacing of hoops ($l = 800$ mm)

The effect of the diameter of hoops on the strength of SRC column is shown in Figure 6.5. The ultimate strength of column with hoops ($\Phi=8$ mm) is larger than the strength of column with 6 mm hoops due to the increased lateral confinement. Compared to a column with steel grade S690, increasing the hoop diameter has a negligible influence on the column strength for a column with steel strength less than or equal to 550 Mpa. Similar to the previous case, for steel grade S690, the increase in steel stress due to lateral confinement contribute to the improvement in column strength.

Figure 6.5 Effect of diameter of hoops

The effect of volumetric ratio of hoops on the ultimate strength of SRC column is shown in Figure 6.6. Similarly, the strength of column increases with an increment of volumetric ratio of hoops. For steel grade S355, S460 and S550, the increase in volumetric ratio of hoops has a smaller effect on the column strength, while the column strength increases considerably by increasing the volumetric ratio from 0.5% to 2.8% for a column with steel grade S690 due to the contribution from increased steel stress.

Therefore, the ultimate strength of column improves by increasing the diameter of hoops and volumetric ratio of hoops, or decreasing of hoop spacing. For column with a high strength steel S690, these parameters have a large effect on the ultimate strength due to contribution from increased confined concrete and steel stress. For a column with steel grade S355, S460 and S550, these parameters have a marginal

effect on the column strength. Additionally, the ductility of SRC column improves due to lateral confinement from hoops and steel section.

(a) S460

(b) S690

Figure 6.6 Effect of volumetric ratio of hoops

6.2 SRC Columns under Eccentric Axial Loading

6.2.1 Parameters

Similar to the SRC columns under concentric axial loading, Parameters including concrete strength, yield strength of structural steel and reinforcement, column

slenderness, eccentricity ratio, spacing and volumetric ratio of transverse reinforcement were considered. The values of parameters are summarized in Table 6.2. A total of 216 SRC columns under eccentric axial loading were analysed in this study. The dimensions and material properties of SRC columns are shown in Table B2 of Appendix B.

Table 6.2 Values of parameters

L (mm)	f_c (MPa)	f_y (MPa)	S (mm)	Eccentric ratio	ρ_{sh} (%)
800	50	355	50	0.1	0.9
1600	70	460	100	0.25	1.4
2400	90	550	150		2.8
		690			

6.2.2 Effect of Parameters on Column strength

6.2.2.1 Effect of Eccentricity

The effect of eccentricity on the strength of SRC columns was plotted against the concrete strength, as shown in Figure 6.7. For SRC column with small eccentricity, $0.1 h_c$, it can be seen that the bearing capacity increases linearly with the increment of concrete strength and steel strength. For column having an eccentricity of $0.25 h_c$, the relationship between column strength and concrete strength is approximately linear for steel grade S355. However, for column with steel grade from S460 to S690, the column strength increases linearly with the increment of concrete strength when $f_c \leq 70\text{MPa}$, while the increment of concrete strength has marginal effect on column strength if $f_c > 70\text{MPa}$. Therefore, it indicates that the effect of material strength on the bearing capacity is significant for column with small eccentricity. For column with higher eccentricity, the effect of material strength on the bearing capacity is significant when $f_c \leq 70\text{MPa}$, while the effect is negligible if $f_c > 70\text{MPa}$.

Figure 6.7 Effect of material strength and eccentricity

6.2.2.2 Effect of Material Strength

Figure 6.8 shows the effect of material strength on the ultimate strength of SRC column. It indicates that the strength of SRC column increases linearly with increasing in concrete strength. The ultimate strength of column increases linearly with increasing steel strength from 355 Mpa to 690 Mpa at a spacing of hoops equal to 50 mm, as shown in Figure 6.8 (a). For steel strength $f_y \leq 550$ Mpa, the ultimate strength of column increases linearly with increase in steel strength. However, for $f_y > 550$ Mpa, increase in the steel strength has a negligible influence on the strength of columns with larger spacing of hoops due to insufficient confinement, as shown in Figure 6.8(b)-(c). This is due to the strain-compatibility problem. The reason has been illustrated in previous section for a column subjected to concentric axial load.

(c) Spacing 150 mm

Figure 6.8 Effect of material strength ($l=800$ mm, $e/h_c=0.1$)

6.2.2.3 Effect of Hoops

The effect of hoop spacing on the column strength is shown in Figure 6.9. It indicates that the ultimate strength of column increases when the hoops spacing decreases. For steel grade S355, S460 and S550, the improvement of column strength is lower than that of steel grade S690. The reason is that concrete crushing and steel yielding occur when the column reaches its ultimate strength for steel strength $f_y \leq 550$ Mpa. The ultimate strength of column increase by increasing in confined concrete strength. However, for steel strength $f_y > 550$ Mpa, early concrete crushing occurs without yielding of steel. With a decrease in spacing of hoops, the lateral confinement level increases. The full strength of steel may be developed due to the increased concrete strain. In this case, the column strength increases as the real stress of confined concrete steel increases due to lateral confinement effect.

The effect of volumetric ratio of hoops on the strength of SRC column is similar to the effect of spacing of hoops. The volumetric ratios are 2.8% (50mm), 1.4% (100mm) and 0.9% (150mm). The bearing capacity of column increases with increasing volumetric ratio of hoops. For steel grade S355, S460 and S550, the increase of volumetric ratio of hoops has a small effect on the column strength, while the column strength increases considerably with increasing the volumetric ratio from 0.9% to 2.8% for column with steel grade S690 due to the contribution of increased steel stress.

It can be concluded that the spacing and volumetric ratio of hoops have a large effect on the bearing capacity due to the contribution of increased concrete confinement and steel stress for column with steel grade S690. And, in columns with steel grade S355, S460 and S550, these parameters have a negligible effect on the column strength.

Figure 6.9 Effect of spacing of hoops ($l = 800$ mm)

6.3 Comparison between FE and Design Methods

In the parametric study, the ultimate strengths of SRC columns were obtained from FE analysis. The resistance of these columns were predicted by two methods. The first one was the plastic method provided by EC4 and the other one was the analytical method proposed in Chapter 4 which considered the strain-compatibility issue and lateral confinement effect from lateral hoops and structural steel section. The detailed calculation is provided in [Appendix C](#). To assess the applicability of

the design methods, the ultimate strengths of SRC columns obtained from FE analysis were compared with the values from EC4 approach and analytical model.

6.3.1 SRC Columns under Concentric Axial Loading

The average strength ratios (N_{FE}/N_{EC4} and N_{FE}/N_P) against steel yield strength are given in Table 6.2. The mean value of N_{FE}/N_{EC4} is 1.06 with the COV of 0.062. The mean value of N_{FE}/N_P is 1.04 with COV of 0.044. It shows that the analytical model considering lateral confinement and strain-compatibility is more accurate for predicting the column strength compared with the full plastic method of EC4. However, the strength ratios are not always greater than 1 for all strength materials although the average ratio for all analysis data is higher than 1.

The strength ratios against steel strength are also plotted in Figure 6.10. The ratios are categorized into four groups based on steel strength (S355, S460, S550 and S690). It is observed that for columns with $f_y \leq 460$ Mpa, all the strength ratios are greater than 1. The mean values of strength ratio N_{FE}/N_{EC4} and N_{FE}/N_P are 1.09 and 1.07 respectively. It is concluded that the strengths of SRC columns obtained from the two methods are conservative. The EC4 method is more conservative than the analytical method because the effect of lateral confinement is not considered in EC4. Similarly, for columns with high strength steel ($f_y \leq 550$ Mpa), the average value of strength ratio for both methods are all greater than 1. Both EC4 methods and analytical model are conservative. However, this is contrary to what is observed for a column with high strength steel S690. The average strength ratio of N_{FE}/N_{EC4} is 0.98, while mean value of N_{FE}/N_P is 1.01. It means that the EC4 using full plastic strength of steel section overestimates the strength of SRC column due to insufficient lateral confinement. The analytical model shows good agreement with the FE value. Thus, it is applicable for predicting the ultimate strength of columns with steel grade up to S690.

Table 6.3 Influence of steel strength on strength ratio for SRC column

		Yield strength of steel			All Data
		$\leq 460 \text{ N/mm}^2$	$\leq 550 \text{ N/mm}^2$	$\leq 690 \text{ N/mm}^2$	
$\frac{N_{FE}}{N_{EC4}}$	Mean	1.09	1.04	0.98	1.06
	COV	0.059	0.038	0.042	0.062
$\frac{N_{FE}}{N_P}$	Mean	1.07	1.02	1.01	1.04
	COV	0.051	0.029	0.027	0.044

Figure 6.10 Comparison of strength ratio against steel strength

6.3.2 SRC Columns under Eccentric Axial Loading

The FE results of SRC columns subjected to eccentric loading were compared with the strength determined from EC4 methods and analytical model. The average strength ratios (N_{FE}/N_{EC4} and N_{FE}/N_P) against steel yield strength are given in [Table 6.4](#). The mean value of N_{FE}/N_{EC4} is 1.07 with the COV of 0.045. The mean value of N_{FE}/N_P is 1.06 with COV of 0.037. It indicates that the analytical model considering lateral confinement and strain-compatibility is more accurate for predicting the column strength compared to the full plastic method given in EC4.

The strength ratios against steel strength are also plotted in [Figure 6.11](#). The ratios are categorized into four groups based on steel strength (S355, S460, S550 and S690). It can be seen that for columns with $f_y \leq 460$ Mpa, all the strength ratios are greater than 1. The mean values of strength ratio N_{FE}/N_{EC4} and N_{FE}/N_P are 1.13 and 1.11 respectively. It is concluded that the strengths obtained from the two methods are conservative. The EC4 method is more conservative than the analytical method because the effect of lateral confinement is not considered in EC4. Similarly, for columns with high strength steel ($f_y \leq 550$ Mpa), the average value of strength ratio for both methods are all greater than 1. Both EC4 and analytical model are conservative. However, it is not the situation for column with high strength steel S690. The average strength ratio of N_{FE}/N_{EC4} is 0.97, while mean value of N_{FE}/N_P is 1.01. It means that the EC4 using full plastic strength of steel overestimates the ultimate strength of SRC column. The analytical model agrees well with the FE value. Thus, it is applicable for calculating the ultimate strength of columns with high strength materials.

Table 6.4 Influence of steel strength on strength ratio for SRC column

		Yield strength of steel			All Data
		$\leq 460 \text{ N/mm}^2$	$\leq 550 \text{ N/mm}^2$	$\leq 690 \text{ N/mm}^2$	
$\frac{N_{FE}}{N_{EC4}}$	Mean	1.13	1.06	0.97	1.07
	COV	0.062	0.057	0.053	0.045
$\frac{N_{FE}}{N_P}$	Mean	1.11	1.05	1.01	1.06
	COV	0.054	0.047	0.029	0.037

(a) S355

(b) S460

(c) S550

(d) S690

Figure 6.11 Comparison of strength ratio against steel strength

Therefore, it is concluded that the plastic method provided in EC4 is applicable for predicting the strength of SRC column with concrete cylinder strength up to 90 N/mm^2 and yield strength of steel up to 550 N/mm^2 . The analytical model considering strain-compatibility and lateral confinement can be used to predict the

strength of SRC column with concrete cylinder strength up to 90 Mpa and steel strength up to 690 Mpa.

Chapter 7 Design Rules on SRC Columns

7.1 Application of EC4 and Analytical Method

Current design code of EC4 is applicable to SRC columns with concrete grades C20/25-C50/60 and steel grades S235-S460. The application is limited to specified range due to the following reasons: (1) insufficient test data on SRC columns with high strength materials; (2) strain-compatibility issue between high strength steel and concrete. In parametric study, a total 432 SRC column models were analysed to predict the strength of SRC columns. Comparison with EC4 outside its range of application was used to assess the applicability of EC4 to cover a wider range of material strength.

Based on the comparison results, the scope of EC4 can be further extended to include concrete cylinder strength $f_c \leq 90 \text{ N/mm}^2$ and yield strength of steel $f_y \leq 550 \text{ N/mm}^2$. For SRC columns with high strength steel $f_y > 550 \text{ Mpa}$, EC4 cannot be applied to predict the column strength due to insufficient confinement. The full strength of steel is not developed in such a case and the plastic method of EC4 overestimates the column strength. It should be noted that EC4 is applicable for steel grade S690 when adequate lateral confinement is provided for the concrete core, as shown in Figure 7.1. The most common approach is to use extra transverse reinforcement. However, this method is impractical as the extra transverse reinforcement can lead to congestion near the beam-column junction.

Figure 7.1 Effect of lateral confinement on steel strength

To address the strain –compatibility issue between high strength steel and concrete, an analytical method was proposed in Chapter 4 with considering the strain-compatibility issue and lateral confinement from transverse reinforcement and steel section. A modified Legeron model was proposed to predict the stress and strain of partially confined concrete and highly confined concrete of SRC column. To evaluate the applicability of the proposed equation, comparison between FE results and analytical method was performed. The comparison results shows that the analytical model can be applied to calculate the strength of SRC column with $f_c \leq 90$ Mpa and $f_c \leq 690$ Mpa.

7.2 Comparison between Original and Alternative Design

As mentioned above, the scope of EC4 can be further extended to include concrete cylinder strength up to 90 N/mm^2 and yield strength of steel up to 550 N/mm^2 . EC4 is applicable for steel grade S690 when adequate lateral confinement is provided for the concrete core. However, it is impractical as the extra transverse reinforcement may lead to congestion. For SRC column with high strength materials, the proposed design procedure is as follows:

- (1) Confirm the dimension of column, steel shape, arrangement of reinforcement bar and materials (concrete class and steel grade);
- (2) Select the spacing of transverse reinforcement where the diameter of links takes a certain value or select the diameter of links where spacing of links takes a certain value;
- (3) Compute the lateral confinement pressures $f_{l,p}$ and $f_{l,h}$;
- (4) Estimate the confined concrete strengths $f_{c,p}$ and $f_{c,h}$ and the corresponding strain $\varepsilon_{c,p}$ and $\varepsilon_{c,h}$;
- (5) Estimate the stress $f_{r,y}$ in the steel;
- (6) Repeat steps (2) to (5) until $f_{r,y}$ reaches the yield strength of steel;

- (7) Estimate the resistance of column in accordance with the procedure given in EC4.

Based on the above design procedure, a comparison between original and alternative design was conducted to study the design methods.

Figure 7.2 shows the original design of SRC columns from a practical project. In the original design, the concrete cylinder strength was 50 MPa and the yield strength of steel and rebar was 355 MPa and 500 MPa respectively. The diameter and spacing of transverse reinforcement (hoops) was 10 mm and 200 mm respectively. The diameter of the longitudinal reinforcement was 25 mm.

Figure 7.2 Original design

The high strength concrete (90 MPa) was proposed as alternative design instead of the original design. Thus concrete amount and reinforcement bar amount are reduced and the arrangement is revised. As a result, the dimension of the column is reduced as shown in Figure 7.3.

Figure 7.3 Alternative design with high strength concrete

The high strength concrete (90 MPa) with high strength steel (690 MPa) was proposed as alternative design instead of the original design. Thus the concrete amount and steel amount were reduced and its arrangement was revised as shown in Figure 7.4. To develop the full strength of steel, the spacing of transverse reinforcement was made smaller than the original design according to the equations from the modified confinement model. The smaller spacing of hoops can lead to congestion which is a practical concern that needs to be addressed.

Figure 7.4 Alternative design with high strength steel and concrete

Figure 7.5 shows the comparison of the concrete, steel and rebar amount of different cross-section. There was a reduction of 47% (1.5 ton per meter) in the amount of concrete by adopting the high strength concrete C90/105. There was a

reduction is 46% (0.3 ton per meter) in the amount of structural steel by adopting the high strength steel S690. However, the transverse reinforcement amount increased as the spacing of hoops was smaller than the original design. The transverse reinforcement increased by 200% (0.03 ton per meter) by adopting S690 steel section, as shown in Figure 7.5 (a). Similarly, for original cross-section N2 and N3, similar results can also be obtained. Concrete and steel amount are reduced significantly by adopting high strength material, but the amount of transverse reinforcement is increased by adopting S690 steel. In summation, the total amount of concrete and steel (structural steel and reinforcing steel) is reduced significantly.

Figure 7.5 Comparison of material amount

As mentioned earlier, to develop the full strength of steel grade S690, the spacing of transverse reinforcement became smaller than the original design. The stress and strain of confined concrete was determined from the modified confinement model

given in Chapter 4. Thus the real stress of steel was determined based on the concrete strain and elastic modulus of steel according to strain-compatibility method. Figure 7.6 shows the effect of spacing and diameter of links on the steel stress of the three types SRC columns (HH1, HH2 and HH3). It indicates that the real stress of steel increases with the increase in the diameter of transverse reinforcement when the spacing of links takes a certain value. The steel stress increases with the decreasing of spacing of links due to insufficient lateral confinement, which has been verified in parametric study. For the three types of SRC columns, if the spacing of hoops is same as the original design (200 mm), the real stress of steel grade S690 is about 600 – 650 Mpa depending on the diameter of hoops. For a large spacing of 400 mm, the real stress of steel grade S690 is 590 – 610 Mpa depending on the diameter of links and the dimension of column.

Figure 7.6 Effect of spacing and diameter of links

The concrete strength also has an effect on the stress in steel according to the strain-compatibility method because the strain of different concrete class is different. Figure 7.7 shows the effect of spacing of links and concrete cylinder strength on the real stress in steel. It indicates that the real stress of steel increases with the increasing of concrete cylinder strength for these three types of SRC columns. This is because that the strain of concrete at peak strength increases with the increasing of concrete cylinder strength. Thus the stress in steel section increases according to the stress-strain relationship of steel even though the lateral confinement pressure is the same. Therefore, for the same arrangement of reinforcement bar, the stress in steel can reach a higher value when a higher strength concrete is used in the SRC columns.

Figure 7.7 Effect of spacing of links and concrete strength

Chapter 8 Conclusions

8.1 Conclusions

In the current design code EC4, for SRC columns the applicability of the design is limited to concrete C20/25-C50/60 and steel S235-S460. The primary purpose of this study was to investigate the strength of SRC columns with high strength materials subjected to concentric and eccentric loading, and to extend the scope of EC4 to fill the gap by allowing SRC columns with concrete strength up to 90 N/mm², steel strength up to 550 N/mm². Literature review indicated that most of the published test data on SRC column focused on normal strength material, very few tests were conducted to utilize the high strength materials. Further research on SRC column with high strength materials needs to be conducted to explore efficient and economical structural designs.

In this study, a comparative study of the confined concrete models (Mander Model, Legeron Model, CEB Model Code 90, FIB Model Code 2010 and EC2) was carried out to determine the most accurate model that was used in design and numerical simulation. An analytical approach was proposed to calculate the strength of SRC columns where consideration was given to strain-compatibility and lateral confinement issues. It was followed by the development of FE model using ABAQUS to simulate the behaviour of SRC columns with high strength materials (structural steel, concrete and reinforcing steel) under both concentric loading and eccentric loading. This model was validated by published test data. Furthermore, a parametric study was carried out to investigate the effects of concrete and steel strength, diameter, spacing and volumetric ratio of lateral reinforcement, eccentricity ratio and slenderness ratio on the behaviour of SRC columns. In the parametric study, the steel and concrete strength were varied from 355 to 690 N/mm² and 50 to 90 N/mm² respectively. The diameter of transverse reinforcement used was 6mm and 8mm respectively, and the spacing of lateral reinforcement was 50mm, 100mm and 150mm respectively. The eccentricity ratio was varied from 0.1 to 0.25 and the column length from 800mm to 2400mm.

The FE results were compared with EC4 to check the applicability of EC4 to SRC columns having high strength materials. In addition, the FE results were also compared with the values from the proposed analytical method to determine the accuracy of the method for predicting the behaviour of SRC column. Based on the study, the following conclusions were obtained:

- (1) The comparative study of available confined concrete models for partially confined concrete indicated that Legeron Model can accurately predict the stress and strain of PCC. A modified Legeron model considering the additional confinement from steel section was proposed for predicting the stress and strain of PCC and HCC in SRC column.
- (2) An analytical method considering strain-compatibility and lateral confinement was proposed for predicting the ultimate strength of SRC columns.
- (3) A FE Model was performed and verified by published test data to investigate the behaviour of SRC columns.
- (4) The ultimate strengths of SRC columns increased linearly with the increment of concrete and steel strength for short columns. However, increasing material strength had a little effect on the ultimate strength of SRC columns for slender columns.
- (5) Ultimate strengths of SRC columns increased when the diameter and volumetric ratio of hoops increased and the spacing of hoops decreased. Compared to a column with normal strength steel, the diameter, spacing and volumetric ratio of hoops had a significant effect on the bearing capacity of column with high strength steel.
- (6) The effect of material strength was significant for column with small eccentricity. For column with higher eccentricity, the effect of material strength was significant for $f_c \leq 70$ Mpa, while the effect was marginal for $f_c > 70$ Mpa.
- (7) The comparison of results between EC4 approach, published test data and FE method indicated that EC4 was conservative for predicting the bearing

capacity of SRC columns with materials strength ($f_c \leq 90 \text{ N/mm}^2$ and $f_y \leq 550 \text{ N/mm}^2$). However, EC4 overestimates the ultimate strength of SRC columns with steel strength higher than 550 N/mm^2 as the plastic method provided in EC4 is not applicable for SRC columns with high strength materials.

- (8) The scope of EC4 can be further extended to cover a wide range of material strength including concrete strength up to 90 N/mm^2 and yield strength of steel up to 550 N/mm^2 .
- (9) The proposed method which considered the strain-compatibility and lateral confinement can accurately predict the strength of SRC columns with concrete strength up to 90 N/mm^2 and steel strength up to 690 N/mm^2 .

8.2 Recommendations for Future Work

In this study, SRC columns with H-shaped steel under concentric axial loading and eccentric axial loading was analysed. Other steel shapes and loading conditions were not considered. Further studies on SRC columns can be conducted in the following areas:

- (1) The effect of steel section shapes (I-, T, Cross-shapes) combined with high strength materials on the behaviour of SRC columns. The lateral confinement pressure is influenced by the steel section shapes in SRC columns. Thus the change in the shape of steel section can lead to a difference in the lateral confinement pressure. A higher strength steel can be used in SRC column design when higher level confinement can be developed.
- (2) Finite element models should be developed for predicting the strength of SRC columns under various conditions (seismic or fire). Finite element model of SRC columns under static loading has been developed. To investigate the behaviour of SRC column under fire or seismic loading, more detailed finite element models should be developed.
- (3) The applicability of EC4 for the fire design of SRC column with high strength material can be investigated. The behaviour of SRC columns with

high strength materials after an exposure to fire can be investigated by experimental study and numerical simulation. These researches can be utilized to assess the applicability of EC4 for fire design of column with high strength material.

- (4) A special way by taking advantage of creep and shrinkage redistribution effects to increase the concrete compressive strain may be investigated. This method does not need to increase lateral hoops. To address the strain-compatibility issue and to fully exploit the compressive yield strength of steel, the creep and shrinkage redistribution may be considered in the design of SRC columns.
- (5) A special type of concrete-Engineered Cementitious Composite (ECC) (which is commonly called “bendable concrete”) to enhance the strength and strain of concrete in SRC column should be investigated. As the traditional approach-the use of extra transverse reinforcement may lead to congestion near beam-column junction which is impractical and the use of external fiber-reinforced polymer (FRP) jacket is still vulnerable to fire attack. Thus ECC as an exterior skin can confine the concrete core and create a cost effective, efficient and high capacity SRC column.

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Appendix A Calculation of Confined Concrete

Test Specimen C1 (Chang-Soo Kim 2013)

Cross-section:

$$h_c = 260 \text{ mm}$$

$$b_c = 260 \text{ mm}$$

$$c = 15 \text{ mm}$$

Steel section:

$$H 150 \times 100 \times 17.6 \times 17.6 \text{ mm}$$

Longitudinal reinforcement:

$$6 \Phi 13$$

Transverse reinforcement:

$$\Phi 10 @ 50 \text{ mm}$$

Material Properties:

Steel section:

$$f_y = 913 \text{ Mpa}$$

$$E_a = 205 \text{ GPa}$$

Concrete:

$$f_c = 94 \text{ Mpa}$$

$$\varepsilon_{co} = 0.0026$$

$$E_c = 38 \text{ Mpa}$$

Longitudinal reinforcement:

$$f_{s,l} = 525 \text{ Mpa}$$

$$E_s = 203 \text{ GPa}$$

Transverse reinforcement:

$$f_{s,t} = 560 \text{ Mpa}$$

$$E_s = 200 \text{ GPa}$$

CEB Model 90 / EC2

Partially Confined Concrete (PCC):

$$\omega_{wd} = \frac{\text{Volume of hoops}}{\text{Volume of confined concrete}} \frac{f_{yd}}{f_{cd}} \quad 0.267$$

$$\alpha_n = 1 - \frac{\sum b_i^2 / 6}{b_0 h_0} \quad 0.56$$

$$\alpha_s = \left(1 - \frac{s}{2b_0}\right) \left(1 - \frac{s}{2h_0}\right) \quad 0.881$$

$$\sigma_2 / f_{ck} = 0.5 \alpha_n \alpha_s \omega_{wd} \quad 0.066$$

$$f_{ck,c} = f_{ck} (1.125 + 2.5 \sigma_2 / f_{ck}) \quad 121 \text{ Mpa}$$

$$\varepsilon_{c2,c} = \varepsilon_{c2} (f_{ck,c} / f_{ck})^2 \quad 0.0044$$

$$\varepsilon_{cu2,c} = \varepsilon_{cu2} + 0.2 \sigma_2 / f_{ck} \quad 0.0158$$

Highly Confined Concrete (HCC):

$$k'_e = \frac{\text{total concrete area between flanges-area of parabolas}}{\text{total concrete area between flanges}} \quad 0.536$$

$$f'_{ls} = k'_e \frac{t^2 f_{r,y}}{3l^2} \quad 29 \text{ Mpa}$$

$$\sigma'_2 / f_{ck} = \sigma_2 / f_{ck} + f'_{ls} / f_{ck} \quad 0.376$$

$$f_{ck,c} = f_{ck} (1.125 + 2.5 \sigma'_2 / f_{ck}) \quad 194 \text{ Mpa}$$

$$\varepsilon_{c2,c} = \varepsilon_{c2} (f_{ck,c} / f_{ck})^2 \quad 0.011$$

$$\varepsilon_{cu2,c} = \varepsilon_{cu2} + 0.2 \sigma'_2 / f_{ck} \quad 0.078$$

Mander Model

Partially Confined Concrete (PCC):

$$k_e = \frac{\left(1 - \sum_{i=1}^n \frac{(w'_i)^2}{6b_c d_c}\right) \left(1 - \frac{s'}{2b_c}\right) \left(1 - \frac{s'}{2d_c}\right)}{1 - \rho_{cc}} \quad 0.446$$

$$\rho_y = \frac{A_{sy}}{s' d_c} \quad 0.015$$

$$f_{ly} = \rho_y f_{yh} \quad 8.37 \text{ Mpa}$$

$$f'_l = k_e f_l \quad 3.74 \text{ Mpa}$$

$$f'_{cc} = f'_{co} \left(-1.254 + 2.254 \sqrt{1 + \frac{7.94 f'_l}{f'_{co}}} - 2 \frac{f'_l}{f'_{co}} \right) \quad 118 \text{ Mpa}$$

$$\varepsilon_{cc} = \varepsilon_{co} \left[1 + 5 \left(\frac{f'_{cc}}{f'_{co}} - 1 \right) \right] \quad 0.006$$

Highly Confined Concrete (HCC):

$$k'_e = \frac{\text{total concrete area between flanges-area of parabolas}}{\text{total concrete area between flanges}} \quad 0.536$$

$$f'_{ls} = k'_e \frac{t^2 f_{r,y}}{3l^2} \quad 29.7 \text{ Mpa}$$

$$f'_l = f'_{l,p} + f'_{ls} \quad 33.4 \text{ Mpa}$$

$$f'_{cc} = f'_{co} \left(-1.254 + 2.254 \sqrt{1 + \frac{7.94 f'_l}{f'_{co}}} - 2 \frac{f'_l}{f'_{co}} \right) \quad 229 \text{ Mpa}$$

$$\varepsilon_{cc} = \varepsilon_{co} \left[1 + 5 \left(\frac{f'_{cc}}{f'_{co}} - 1 \right) \right] \quad 0.0216$$

Fib Model Code 2010

Partially Confined Concrete (PCC):

$$\alpha = \left(1 - \frac{\sum b_i^2/6}{b_c h_c}\right) \left(1 - \frac{s}{b_c}\right) \left(1 - \frac{s}{h_c}\right) \quad 0.422$$

$$\omega_c = \min \left\{ \omega_y = \frac{A_{sy} f_{yd}}{b_c s f_{cd}}, \omega_z = \frac{A_{sz} f_{yd}}{h_c s f_{cd}} \right\} \quad 0.14$$

$$\sigma_2 = \alpha \omega_c f_{cd} \quad 3.37 \text{ Mpa}$$

$$f_{ck,c} = f_{ck} \left(1.0 + 3.5 \left(\frac{\sigma_2}{f_{ck}} \right)^{0.75} \right) \quad 122 \text{ Mpa}$$

$$\varepsilon_{c,c} = \varepsilon_c \left[1 + 5 \left(\frac{f_{ck,c}}{f_{ck}} - 1 \right) \right] \quad 0.0066$$

$$\varepsilon_{cu2,c} = \varepsilon_{cu2} + 0.2 \sigma_2 / f_{ck} \quad 0.01$$

Highly Confined Concrete (HCC):

$$k'_e = \frac{\text{total concrete area between flanges-area of parabolas}}{\text{total concrete area between flanges}} \quad 0.536$$

$$f'_{ls} = k'_e \frac{t^2 f_{r,y}}{3l^2} \quad 29.7 \text{ Mpa}$$

$$\sigma'_2 = \sigma_2 + f'_{ls} \quad 33.3 \text{ Mpa}$$

$$f_{ck,c} = f_{ck} \left(1.0 + 3.5 \left(\frac{\sigma'_2}{f_{ck}} \right)^{0.75} \right) \quad 244 \text{ Mpa}$$

$$\varepsilon_{c,c} = \varepsilon_c \left[1 + 5 \left(\frac{f_{ck,c}}{f_{ck}} - 1 \right) \right] \quad 0.0237$$

$$\varepsilon_{cu2,c} = \varepsilon_{cu2} + 0.2 \sigma'_2 / f_{ck} \quad 0.0734$$

Legeron Model

Partially Confined Concrete (PCC):

$$k_e = \frac{\left(1 - \sum_{i=1}^n \frac{(w'_i)^2}{6b_c d_c}\right) \left(1 - \frac{s'}{2b_c}\right) \left(1 - \frac{s'}{2d_c}\right)}{1 - \rho_{cc}} \quad 0.446$$

$$\rho_s = \frac{A_{sh}}{s' d_c} \quad 0.03$$

$$\rho_{se} = k_e \rho_s \quad 0.0067$$

$$\kappa = \frac{f'_c}{\rho_{se} E_s \varepsilon'_c} \quad 26.43$$

$$f'_h = \begin{cases} f_h & \kappa \leq 10 \\ \frac{0.25 f'_c}{\rho_{se} (\kappa - 10)} \geq 0.43 \varepsilon'_c E_s & \kappa \geq 10 \end{cases} \quad 223.6 \text{ Mpa}$$

$$f'_{le} = \rho_{se} f'_h \quad 1.49 \text{ Mpa}$$

$$I'_e = \frac{f'_{le}}{f'_c} \quad 0.016$$

$$f'_{cc} = f'_c \left(1 + 2.4 (I'_e)^{0.7}\right) \quad 106 \text{ Mpa}$$

$$\varepsilon'_{cc} = \varepsilon'_c \left(1 + 35 (I'_e)^{1.2}\right) \quad 0.0032$$

Highly Confined Concrete (HCC):

$$k'_e = \frac{\text{total concrete area between flanges-area of parabolas}}{\text{total concrete area between flanges}} \quad 0.536$$

$$f'_{ls} = k'_e \frac{t^2 f_{r,y}}{3l^2} \quad 21.58 \text{ Mpa}$$

$$f'_l = f'_{l,p} + f'_{ls} \quad 23 \text{ Mpa}$$

$$I'_e = \frac{f'_{le}}{f'_c} \quad 0.245$$

$$f'_{cc} = f'_c \left(1 + 2.4 (I'_e)^{0.7} \right) \quad 178 \text{ Mpa}$$

$$\varepsilon'_{cc} = \varepsilon'_c \left(1 + 35 (I'_e)^{1.2} \right) \quad 0.0146$$

Appendix B Parametric Study

Table B1 Specimen dimensions and material properties of SRC columns

Specimen		Dimension			Structural steel section (mm)	Reinforcement			Material			
		b_c	h_c	L		Long.	Transverse		f_c	f_a	$f_{s,l}$	$f_{s,h}$
		(mm)	(mm)	(mm)			Φ	S	(MPa)	(MPa)	(MPa)	(MPa)
G1	1	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	50	355	500	500
	2	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	70	355	500	500
	3	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	90	355	500	500
	4	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	50	460	500	500
	5	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	70	460	500	500
	6	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	90	460	500	500
	7	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	50	550	500	500
	8	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	70	550	500	500
	9	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	90	550	500	500
	10	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	50	690	500	500
	11	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	70	690	500	500
	12	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	90	690	500	500

G2	13	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	50	355	500	500
	14	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	70	355	500	500
	15	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	90	355	500	500
	16	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	50	460	500	500
	17	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	70	460	500	500
	18	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	90	460	500	500
	19	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	50	550	500	500
	20	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	70	550	500	500
	21	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	90	550	500	500
	22	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	50	690	500	500
	23	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	70	690	500	500
	24	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	90	690	500	500
G3	25	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	50	355	500	500
	26	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	70	355	500	500
	27	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	90	355	500	500
	28	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	50	460	500	500
	29	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	70	460	500	500
	30	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	90	460	500	500
	31	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	50	550	500	500

	32	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	150	70	550	500	500
	33	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	150	90	550	500	500
	34	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	150	50	690	500	500
	35	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	150	70	690	500	500
	36	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	150	90	690	500	500
G4	37	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	50	355	500	500
	38	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	70	355	500	500
	39	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	90	355	500	500
	40	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	50	460	500	500
	41	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	70	460	500	500
	42	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	90	460	500	500
	43	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	50	550	500	500
	44	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	70	550	500	500
	45	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	90	550	500	500
	46	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	50	690	500	500
	47	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	70	690	500	500
	48	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	90	690	500	500
G5	49	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	100	50	355	500	500
	50	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	100	70	355	500	500

	51	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	90	355	500	500
	52	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	50	460	500	500
	53	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	70	460	500	500
	54	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	90	460	500	500
	55	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	50	550	500	500
	56	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	70	550	500	500
	57	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	90	550	500	500
	58	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	50	690	500	500
	59	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	70	690	500	500
	60	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	90	690	500	500
G6	61	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	50	355	500	500
	62	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	70	355	500	500
	63	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	90	355	500	500
	64	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	50	460	500	500
	65	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	70	460	500	500
	66	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	90	460	500	500
	67	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	50	550	500	500
	68	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	70	550	500	500
	69	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	90	550	500	500

	70	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	50	690	500	500
	71	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	70	690	500	500
	72	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	90	690	500	500
G7	73	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	50	355	500	500
	74	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	70	355	500	500
	75	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	90	355	500	500
	76	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	50	460	500	500
	77	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	70	460	500	500
	78	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	90	460	500	500
	79	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	50	550	500	500
	80	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	70	550	500	500
	81	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	90	550	500	500
	82	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	50	690	500	500
	83	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	70	690	500	500
	84	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	90	690	500	500
G8	85	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	50	355	500	500
	86	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	70	355	500	500
	87	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	90	355	500	500
	88	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	50	460	500	500

	89	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	100	70	460	500	500
	90	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	100	90	460	500	500
	91	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	100	50	550	500	500
	92	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	100	70	550	500	500
	93	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	100	90	550	500	500
	94	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	100	50	690	500	500
	95	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	100	70	690	500	500
	96	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	100	90	690	500	500
G9	97	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	150	50	355	500	500
	98	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	150	70	355	500	500
	99	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	150	90	355	500	500
	100	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	150	50	460	500	500
	101	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	150	70	460	500	500
	102	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	150	90	460	500	500
	103	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	150	50	550	500	500
	104	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	150	70	550	500	500
	105	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	150	90	550	500	500
	106	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	150	50	690	500	500
	107	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	150	70	690	500	500

	108	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	150	90	690	500	500
G10	109	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	50	50	355	500	500
	110	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	50	70	355	500	500
	111	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	50	90	355	500	500
	112	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	50	50	460	500	500
	113	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	50	70	460	500	500
	114	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	50	90	460	500	500
	115	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	50	50	550	500	500
	116	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	50	70	550	500	500
	117	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	50	90	550	500	500
	118	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	50	50	690	500	500
	119	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	50	70	690	500	500
	120	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	50	90	690	500	500
G11	121	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	100	50	355	500	500
	122	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	100	70	355	500	500
	123	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	100	90	355	500	500
	124	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	100	50	460	500	500
	125	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	100	70	460	500	500
	126	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	100	90	460	500	500

	127	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	100	50	550	500	500
	128	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	100	70	550	500	500
	129	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	100	90	550	500	500
	130	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	100	50	690	500	500
	131	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	100	70	690	500	500
	132	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	100	90	690	500	500
G12	133	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	150	50	355	500	500
	134	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	150	70	355	500	500
	135	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	150	90	355	500	500
	136	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	150	50	460	500	500
	137	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	150	70	460	500	500
	138	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	150	90	460	500	500
	139	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	150	50	550	500	500
	140	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	150	70	550	500	500
	141	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	150	90	550	500	500
	142	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	150	50	690	500	500
	143	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	150	70	690	500	500
	144	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	150	90	690	500	500
G13	145	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	50	50	355	500	500

	146	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	50	70	355	500	500
	147	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	50	90	355	500	500
	148	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	50	50	460	500	500
	149	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	50	70	460	500	500
	150	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	50	90	460	500	500
	151	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	50	50	550	500	500
	152	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	50	70	550	500	500
	153	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	50	90	550	500	500
	154	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	50	50	690	500	500
	155	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	50	70	690	500	500
G14	156	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	50	90	690	500	500
	157	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	100	50	355	500	500
	158	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	100	70	355	500	500
	159	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	100	90	355	500	500
	160	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	100	50	460	500	500
	161	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	100	70	460	500	500
	162	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	100	90	460	500	500
	163	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	100	50	550	500	500
	164	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	100	70	550	500	500

	165	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	100	90	550	500	500
	166	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	100	50	690	500	500
	167	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	100	70	690	500	500
	168	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	100	90	690	500	500
G15	169	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	150	50	355	500	500
	170	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	150	70	355	500	500
	171	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	150	90	355	500	500
	172	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	150	50	460	500	500
	173	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	150	70	460	500	500
	174	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	150	90	460	500	500
	175	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	150	50	550	500	500
	176	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	150	70	550	500	500
	177	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	150	90	550	500	500
	178	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	150	50	690	500	500
	179	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	150	70	690	500	500
	180	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	150	90	690	500	500
G16	181	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	50	50	355	500	500
	182	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	50	70	355	500	500
	183	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	6	50	90	355	500	500

	184	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	50	50	460	500	500
	185	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	50	70	460	500	500
	186	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	50	90	460	500	500
	187	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	50	50	550	500	500
	188	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	50	70	550	500	500
	189	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	50	90	550	500	500
	190	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	50	50	690	500	500
	191	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	50	70	690	500	500
	192	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	50	90	690	500	500
G17	193	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	100	50	355	500	500
	194	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	100	70	355	500	500
	195	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	100	90	355	500	500
	196	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	100	50	460	500	500
	197	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	100	70	460	500	500
	198	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	100	90	460	500	500
	199	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	100	50	550	500	500
	200	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	100	70	550	500	500
	201	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	100	90	550	500	500
	202	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	100	50	690	500	500

	203	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	100	70	690	500	500
	204	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	100	90	690	500	500
G18	205	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	150	50	355	500	500
	206	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	150	70	355	500	500
	207	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	150	90	355	500	500
	208	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	150	50	460	500	500
	209	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	150	70	460	500	500
	210	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	150	90	460	500	500
	211	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	150	50	550	500	500
	212	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	150	70	550	500	500
	213	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	150	90	550	500	500
	214	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	150	50	690	500	500
	215	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	150	70	690	500	500
	216	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	6	150	90	690	500	500

Table A2 Dimension and material properties of SRC columns (eccentric loading)

Specimen		Dimension			Structural steel section (mm)	Reinforcement			Eccentricity (mm)	Material			
		b_c (mm)	h_c (mm)	L (mm)		Long.	Transverse			f_c (MPa)	f_a (MPa)	$f_{s,l}$ (MPa)	$f_{s,h}$ (MPa)
								Φ					
G1	1	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	50	355	500	500
	2	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	70	355	500	500
	3	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	90	355	500	500
	4	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	50	460	500	500
	5	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	70	460	500	500
	6	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	90	460	500	500
	7	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	50	550	500	500
	8	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	70	550	500	500
	9	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	90	550	500	500
	10	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	50	690	500	500
	11	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	70	690	500	500
	12	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	90	690	500	500
G2	13	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	100	20	50	355	500	500
	14	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	100	20	70	355	500	500

	15	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	20	90	355	500	500
	16	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	20	50	460	500	500
	17	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	20	70	460	500	500
	18	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	20	90	460	500	500
	19	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	20	50	550	500	500
	20	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	20	70	550	500	500
	21	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	20	90	550	500	500
	22	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	20	50	690	500	500
	23	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	20	70	690	500	500
	24	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	20	90	690	500	500
G3	25	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	20	50	355	500	500
	26	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	20	70	355	500	500
	27	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	20	90	355	500	500
	28	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	20	50	460	500	500
	29	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	20	70	460	500	500
	30	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	20	90	460	500	500
	31	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	20	50	550	500	500
	32	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	20	70	550	500	500
	33	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	20	90	550	500	500

	34	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	150	20	50	690	500	500
	35	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	150	20	70	690	500	500
	36	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	150	20	90	690	500	500
G4	37	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	50	355	500	500
	38	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	70	355	500	500
	39	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	90	355	500	500
	40	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	50	460	500	500
	41	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	70	460	500	500
	42	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	90	460	500	500
	43	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	50	550	500	500
	44	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	70	550	500	500
	45	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	90	550	500	500
	46	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	50	690	500	500
	47	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	70	690	500	500
	48	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	90	690	500	500
G5	49	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	100	20	50	355	500	500
	50	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	100	20	70	355	500	500
	51	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	100	20	90	355	500	500
	52	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	100	20	50	460	500	500

	53	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	20	70	460	500	500
	54	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	20	90	460	500	500
	55	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	20	50	550	500	500
	56	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	20	70	550	500	500
	57	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	20	90	550	500	500
	58	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	20	50	690	500	500
	59	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	20	70	690	500	500
	60	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	20	90	690	500	500
G6	61	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	20	50	355	500	500
	62	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	20	70	355	500	500
	63	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	20	90	355	500	500
	64	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	20	50	460	500	500
	65	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	20	70	460	500	500
	66	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	20	90	460	500	500
	67	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	20	50	550	500	500
	68	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	20	70	550	500	500
	69	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	20	90	550	500	500
	70	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	20	50	690	500	500
	71	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	20	70	690	500	500

	72	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	150	20	90	690	500	500
G7	73	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	50	355	500	500
	74	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	70	355	500	500
	75	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	90	355	500	500
	76	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	50	460	500	500
	77	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	70	460	500	500
	78	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	90	460	500	500
	79	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	50	550	500	500
	80	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	70	550	500	500
	81	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	90	550	500	500
	82	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	50	690	500	500
	83	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	70	690	500	500
	84	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	20	90	690	500	500
G8	85	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	100	20	50	355	500	500
	86	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	100	20	70	355	500	500
	87	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	100	20	90	355	500	500
	88	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	100	20	50	460	500	500
	89	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	100	20	70	460	500	500
	90	200	200	2400	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	100	20	90	460	500	500

	91	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	20	50	550	500	500
	92	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	20	70	550	500	500
	93	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	20	90	550	500	500
	94	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	20	50	690	500	500
	95	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	20	70	690	500	500
	96	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	20	90	690	500	500
G9	97	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	20	50	355	500	500
	98	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	20	70	355	500	500
	99	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	20	90	355	500	500
	100	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	20	50	460	500	500
	101	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	20	70	460	500	500
	102	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	20	90	460	500	500
	103	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	20	50	550	500	500
	104	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	20	70	550	500	500
	105	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	20	90	550	500	500
	106	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	20	50	690	500	500
	107	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	20	70	690	500	500
	108	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	20	90	690	500	500
G10	109	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	50	50	355	500	500

	110	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	50	70	355	500	500
	111	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	50	90	355	500	500
	112	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	50	50	460	500	500
	113	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	50	70	460	500	500
	114	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	50	90	460	500	500
	115	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	50	50	550	500	500
	116	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	50	70	550	500	500
	117	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	50	90	550	500	500
	118	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	50	50	690	500	500
	119	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	50	70	690	500	500
	120	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	50	90	690	500	500
G11	121	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	50	50	355	500	500
	122	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	50	70	355	500	500
	123	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	50	90	355	500	500
	124	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	50	50	460	500	500
	125	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	50	70	460	500	500
	126	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	50	90	460	500	500
	127	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	50	50	550	500	500
	128	200	200	800	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	50	70	550	500	500

	129	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	100	50	90	550	500	500
	130	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	100	50	50	690	500	500
	131	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	100	50	70	690	500	500
	132	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	100	50	90	690	500	500
G12	133	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	150	50	50	355	500	500
	134	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	150	50	70	355	500	500
	135	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	150	50	90	355	500	500
	136	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	150	50	50	460	500	500
	137	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	150	50	70	460	500	500
	138	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	150	50	90	460	500	500
	139	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	150	50	50	550	500	500
	140	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	150	50	70	550	500	500
	141	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	150	50	90	550	500	500
	142	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	150	50	50	690	500	500
	143	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	150	50	70	690	500	500
	144	200	200	800	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	150	50	90	690	500	500
G13	145	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	50	50	355	500	500
	146	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	50	70	355	500	500
	147	200	200	1600	$100 \times 100 \times 6 \times 8$	4 $\Phi 10$	8	50	50	90	355	500	500

	148	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ10	8	50	50	50	460	500	500
	149	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ10	8	50	50	70	460	500	500
	150	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ10	8	50	50	90	460	500	500
	151	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ10	8	50	50	50	550	500	500
	152	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ10	8	50	50	70	550	500	500
	153	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ10	8	50	50	90	550	500	500
	154	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ10	8	50	50	50	690	500	500
	155	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ10	8	50	50	70	690	500	500
	156	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ10	8	50	50	90	690	500	500
G14	157	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ10	8	100	50	50	355	500	500
	158	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ10	8	100	50	70	355	500	500
	159	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ10	8	100	50	90	355	500	500
	160	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ10	8	100	50	50	460	500	500
	161	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ10	8	100	50	70	460	500	500
	162	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ10	8	100	50	90	460	500	500
	163	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ10	8	100	50	50	550	500	500
	164	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ10	8	100	50	70	550	500	500
	165	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ10	8	100	50	90	550	500	500
	166	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ10	8	100	50	50	690	500	500

	167	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	50	70	690	500	500
	168	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	50	90	690	500	500
G15	169	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	50	50	355	500	500
	170	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	50	70	355	500	500
	171	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	50	90	355	500	500
	172	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	50	50	460	500	500
	173	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	50	70	460	500	500
	174	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	50	90	460	500	500
	175	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	50	50	550	500	500
	176	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	50	70	550	500	500
	177	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	50	90	550	500	500
	178	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	50	50	690	500	500
	179	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	50	70	690	500	500
	180	200	200	1600	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	50	90	690	500	500
G16	181	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	50	50	355	500	500
	182	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	50	70	355	500	500
	183	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	50	90	355	500	500
	184	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	50	50	460	500	500
	185	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	50	70	460	500	500

	186	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	50	90	460	500	500
	187	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	50	50	550	500	500
	188	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	50	70	550	500	500
	189	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	50	90	550	500	500
	190	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	50	50	690	500	500
	191	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	50	70	690	500	500
	192	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	50	50	90	690	500	500
G17	193	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	50	50	355	500	500
	194	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	50	70	355	500	500
	195	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	50	90	355	500	500
	196	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	50	50	460	500	500
	197	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	50	70	460	500	500
	198	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	50	90	460	500	500
	199	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	50	50	550	500	500
	200	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	50	70	550	500	500
	201	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	50	90	550	500	500
	202	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	50	50	690	500	500
	203	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	50	70	690	500	500
	204	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	100	50	90	690	500	500

G18	205	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	50	50	355	500	500
	206	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	50	70	355	500	500
	207	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	50	90	355	500	500
	208	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	50	50	460	500	500
	209	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	50	70	460	500	500
	210	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	50	90	460	500	500
	211	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	50	50	550	500	500
	212	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	50	70	550	500	500
	213	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	50	90	550	500	500
	214	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	50	50	690	500	500
	215	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	50	70	690	500	500
	216	200	200	2400	$100 \times 100 \times 6 \times 8$	4 Φ 10	8	150	50	90	690	500	500

(a) S355, $\Phi = 6$ mm, $S = 50$ mm

(b) S460, $\Phi = 6$ mm, $S = 50$ mm

(c) S550, $\Phi = 6$ mm, $S = 50$ mm

(d) S690, $\Phi = 6$ mm, $S = 50$ mm

(e) S355, $\Phi = 6$ mm, $S = 100$ mm

(f) S460, $\Phi = 6$ mm, $S = 100$ mm

(g) S550, $\Phi = 6$ mm, $S = 100$ mm

(h) S690, $\Phi = 6$ mm, $S = 100$ mm

(e) S355, $\Phi = 6$ mm, $S = 150$ mm

(f) S460, $\Phi = 6$ mm, $S = 150$ mm

(g) S550, $\Phi = 6$ mm, $S = 150$ mm

(h) S690, $\Phi = 6$ mm, $S = 150$ mm

Figure B1 Effect of Length and concrete strength on column strength

(a) C50, $\Phi = 6$ mm, $S = 50$ mm

(b) C70, $\Phi = 6$ mm, $S = 50$ mm

(c) C90, $\Phi = 6$ mm, $S = 50$ mm

(d) C50, $\Phi = 6$ mm, $S = 100$ mm

(e) C70, $\Phi = 6$ mm, $S = 100$ mm

(f) C90, $\Phi = 6$ mm, $S = 100$ mm

(e) C50, $\Phi = 6$ mm, $S = 150$ mm

(f) C70, $\Phi = 6$ mm, $S = 150$ mm

(f) C90, $\Phi = 6$ mm, $S = 150$ mm

Figure B2 Effect of Length and steel strength on column strength

(a) $\Phi = 8$ mm, $S = 50$ mm $L = 1600$ mm

(b) $\Phi = 8$ mm, $S = 100$ mm $L = 1600$ mm

(c) $\Phi = 8$ mm, $S = 150$ mm $L = 1600$ mm (d) $\Phi = 6$ mm, $S = 50$ mm $L = 800$ mm

(b) $\Phi = 6$ mm, $S = 100$ mm $L = 800$ mm (b) $\Phi = 6$ mm, $S = 150$ mm $L = 800$ mm

Figure B3 Effect of spacing of links and column length on column strength

Appendix C Calculation of SRC Columns

Cross-section		
Depth $h_c =$	200	mm
Width $b_c =$	200	mm
Column length $L =$	800	mm
Steel section H 100 ×100 ×6 ×8		
Yield strength $f_y =$	690	MPa
Elastic modulus $E_a =$	210	GPa
Depth $h =$	100	mm
Width $b =$	100	mm
Flange thickness $t_f =$	8	mm
Web thickness $t_w =$	6	mm
Area $A_a =$	2104	mm ²
$I_{ay} =$	368	cm ⁴
$I_{az} =$	133	cm ⁴
Concrete		
Cylinder strength $f_{ck} =$	90	MPa
Strain at peak $\varepsilon_c =$	0.0028	
Ultimate strain $\varepsilon_{cu} =$	0.0028	
Secant modulus of elasticity $E_{cm} =$	44	GPa
Area $A_c =$	37582	mm ²
$I_{cy} =$	12824	cm ⁴
$I_{cz} =$	13059	cm ⁴
Longitudinal Reinforcement		
Diameter $d =$	10	mm
Number $n =$	4	
Area $A_s =$	314	mm ²
Yield strength $f_s =$	500	MPa
Elastic modulus $E_s =$	210	GPa
$I_{sy} =$	141	cm ⁴
$I_{sz} =$	141	cm ⁴
Transverse reinforcement (hoops)		
Diameter $d =$	6	mm
Spacing $S =$	100	mm
Yield strength $f_{sh} =$	500	MPa

EC4 Approach

Ratio of reinforcement		
$\rho = A_s / A_c$	0.84%	
Plastic resistance to compression		
$N_{pl,Rd} = A_a f_{yd} + 0.85 A_c f_{cd} + A_s f_{sd}$	3505	kN
Steel contribution factor		
$\delta = A_a f_{yd} / N_{pl,Rd} \quad (0.2 \leq \delta \leq 0.9)$	0.41	
Effective elastic flexural stiffness		
About the major axis (y-y):		
$(EI)_{ey} = E_a I_{ay} + 0.6 E_{cm} I_{cy} + E_s I_{sy}$	4.43E+03	kNm ²
About the major axis (z-z):		
$(EI)_{ez} = E_a I_{az} + 0.6 E_{cm} I_{cz} + E_s I_{sz}$	4.0E+03	kNm ²
Elastic buckling load:		
About the major axis (y-y):		
$N_{cry} = \frac{\pi^2 (EI)_{ey}}{L_{ey}^2}$	6.83E+03	kN
About the minor axis (z-z):		
$N_{crz} = \frac{\pi^2 (EI)_{ez}}{L_{ez}^2}$	6.14E+03	kN
Plastic resistance to compression:		
$N_{pl,Rk} = A_a f_y + 0.85 A_c f_{ck} + A_s f_{sk}$	4484	kN
Non-dimensional slenderness ration:		
About the major axis (y-y):		
$\bar{\lambda}_y = \sqrt{\frac{N_{pl,Rk}}{N_{cry}}}$	0.26	
About the minor axis (z-z):		
$\bar{\lambda}_z = \sqrt{\frac{N_{pl,Rk}}{N_{crz}}}$	0.27	
curve b for major axis and strut curve c for minor axis		
About the major axis (y-y):		
$\Phi_y = 0.5 \left(1 + \alpha (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2 \right)$	0.54	

$\chi_y = \frac{1}{\Phi_y + \sqrt{\Phi_y^2 - \lambda_y^2}}$	0.98	
About the minor axis (z-z):		
$\Phi_z = 0.5 \left(1 + \alpha \left(\overline{\lambda_z} - 0.2 \right) + \overline{\lambda_z^2} \right)$	0.55	
$\chi_z = \frac{1}{\Phi_z + \sqrt{\Phi_z^2 - \lambda_z^2}}$	0.96	
Resistance of SRC columns under axial compression		
$N_{Rd} = \chi N_{pl,Rd}$	3380	kN

Analytical Method

Confined concrete		
Partially Confined Concrete (PCC):		
$k_e = \frac{\left(1 - \sum_{i=1}^n \frac{(b_i)^2}{6b_0h_0}\right) \left(1 - \frac{s}{2b_0}\right) \left(1 - \frac{s}{2h_0}\right)}{1 - \rho_s}$	0.223	
$\rho_{sh} = \frac{A_{sh}}{sh_0}$	0.76%	
$\kappa = \frac{f_{c,u}}{k_e \rho_{sh} E_s \varepsilon_c}$	184	
$f_{r,h} = \begin{cases} f_{s,h} & \kappa \leq 10 \\ \frac{0.25 f_{c,u}}{k_e \rho_{sh} (\kappa - 10)} \geq 0.43 \varepsilon_c E_s & \kappa \geq 10 \end{cases}$	241	
$f_{l,p} = k_e \rho_{sh} f_{r,h}$	0.205	
$f_{c,p} = K_{p,l} f_{c,u} = \left(1 + 2.4 \left(\frac{f_{l,p}}{f_{c,u}}\right)^{0.7}\right) f_{c,u}$	93	MPa
$\varepsilon_{c,p} = K_{p,ll} \varepsilon_c = \left(1 + 35 \left(\frac{f_{l,p}}{f_{c,u}}\right)^{1.2}\right) \varepsilon_c$	0.0029	
$f_{r,y}$	588	MPa
Highly Confined Concrete (HCC):		
$k'_e = \frac{\text{total concrete area between flanges-area of parabolas}}{\text{total concrete area between flanges}}$	0.7	
$k_a = \frac{t_f^2}{3l^2}$	0.01	
$f_{l,s} = k'_e k_a f_{r,y}$	4.1	
$f_{l,h} = f_{l,p} + f_{l,s}$	4.3	
$f_{c,h} = K_{h,l} f_{c,u} = \left(1 + 2.4 \left(\frac{f_{l,h}}{f_{c,u}}\right)^{0.7}\right) f_{c,u}$	115	
$\varepsilon_{c,h} = K_{h,ll} \varepsilon_c = \left(1 + 35 \left(\frac{f_{l,h}}{f_{c,u}}\right)^{1.2}\right) \varepsilon_c$	0.0046	
Ratio of reinforcement		

$\rho = A_s / A_c$	0.84%	
Plastic resistance to compression		
$N_{p,Rd} = A_a f_{r,yd} + 0.85(A_{cu} f_{cd,u} + A_{cp} f_{cd,p} + A_{ch} f_{cd,h}) + A_s f_{r,sd}$	3371	kN
Steel contribution factor		
$\delta = A_a f_{yd} / N_{pl,Rd} \quad (0.2 \leq \delta \leq 0.9)$	0.37	
Effective elastic flexural stiffness		
About the major axis (y-y):		
$(EI)_{ey} = E_a I_{ay} + 0.6 E_{cm} I_{cy} + E_s I_{sy}$	4.47E+03	kNm ²
About the major axis (z-z):		
$(EI)_{ez} = E_a I_{az} + 0.6 E_{cm} I_{cz} + E_s I_{sz}$	4.04E+03	kNm ²
Elastic buckling load:		
About the major axis (y-y):		
$N_{cry} = \frac{\pi^2 (EI)_{ey}}{L_{ey}^2}$	6.89E+03	kN
About the minor axis (z-z):		
$N_{crz} = \frac{\pi^2 (EI)_{ez}}{L_{ez}^2}$	6.22E+03	kN
Plastic resistance to compression:		
$N_{pl,Rk} = A_a f_y + 0.85 A_c f_{ck} + A_s f_{sk}$	4391	kN
Non-dimensional slenderness ration:		
About the major axis (y-y):		
$\bar{\lambda}_y = \sqrt{\frac{N_{pl,Rk}}{N_{cry}}}$	0.25	
About the minor axis (z-z):		
$\bar{\lambda}_z = \sqrt{\frac{N_{pl,Rk}}{N_{crz}}}$	0.27	
curve b for major axis and strut curve c for minor axis		
About the major axis (y-y):		
$\Phi_y = 0.5 \left(1 + \alpha (\bar{\lambda}_y - 0.2) + \bar{\lambda}_y^2 \right)$	0.54	
$\chi_y = \frac{1}{\Phi_y + \sqrt{\Phi_y^2 - \bar{\lambda}_y^2}}$	0.98	
About the minor axis (z-z):		

$\Phi_z = 0.5 \left(1 + \alpha \left(\overline{\lambda}_z - 0.2 \right) + \overline{\lambda}_z^2 \right)$	0.55	
$\chi_z = \frac{1}{\Phi_z + \sqrt{\Phi_z^2 - \overline{\lambda}_z^2}}$	0.96	
Resistance of SRC columns under axial compression		
$N_p = \chi N_{p,Rd}$	3259	kN