

Behaviour of Hybrid Tubular Sections Utilising Seawater and Sea Sand Concrete, Fiber Reinforced Polymer and Stainless Steel

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Abstract

With the increase of population, huge demand on ordinary Portland cement concrete is exacerbating the resource shortage (e.g., fresh water, river sand) and causing serious environment impacts (e.g., emission of CO₂ during the production of Portland cement). In this research, the alkali-activated slag concrete with seawater and sea sand is used to replace ordinary Portland cement concrete. Because of the corrosion effect caused by the Chloride in seawater and sea sand, conventional carbon steel is not suitable. Therefore, fiber reinforced polymer (FRP) and stainless steel (SS) are adopted in the hybrid sections due to their greater corrosion resistance.

The present PhD thesis focuses on the behaviour of hybrid tubular sections utilising seawater and sea sand concrete (SWSSC), glass/carbon/basalt fiber reinforced polymer (G/C/B-FRP) and stainless steel (SS). Both the short-term structural behaviour and durability performance were investigated by the means of experimental study and theoretical analysis.

Concrete mixture was firstly proposed for SWSSC which could reach the target strength and desirable workability. Both mechanical and thermal properties of SWSSC exposed to elevated temperatures were experimentally investigated. The Chloride in seawater and sea sand does not have an obvious detrimental effect on the properties of SWSSC. The response of SWSSC subjected to elevated temperatures is similar to that of ordinary concrete, in which the incompatibility of thermal expansion between paste and aggregate is the main reason for the residual strength decrease.

Axial compressive test was conducted on SWSSC-filled FRP and SS tubular stub columns, including both fully filled and double-skin tubes, to study the short-term structural behaviour. The effects of key parameters, such as material type, cross-section dimension, confinement effect, void ratio, on the structural behaviour of columns were extensively investigated. Theoretical models were then proposed to estimate the load-axial strain curves of SWSSC fully filled and double-skin FRP tubes under axial compression. The proposed models could properly account for the influence of biaxial stress state, Poisson effect, buckling of FRP tube, void ratio and non-uniform confinement in double-skin tubes. A comparison between the predicted and experimental curves verified the reasonability and accuracy of the proposed models. Furthermore, design formulas were proposed to estimate the ultimate capacity of SWSSC-filled SS tubes (both fully filled and double-skin) under axial compression.

In order to understand the long-term performance of SWSSC-filled FRP tubes, accelerated degradation test was conducted on SWSSC, FRP and SWSSC-filled FRP tubes. A strength increase was observed for SWSSC cured in artificial seawater (i.e., 3.5% salt solution) due to the further geo-polymerization of SWSSC. If adopting the residual hoop strength as an indicator of the long-term

performance of FRP, CFRP behaved better than GFRP and BFRP. The normalised strength of SWSSC-filled FRP tubes decreased after exposing to 40 °C artificial seawater for up to 6 months, which is mainly caused by the degradation of FRP tubes. Finally, design suggestions were given for the hybrid sections applied in a marine environment. Future research is pointed out in terms of high performance SWSSC, influence of fiber orientation, structural members, long-term field testing and combined effect of sustained loads and environmental aging.

Declaration

I hereby declare that this thesis contains no material which has been accepted for the award of any other degree or diploma at any university or equivalent institution and that, to the best of my knowledge and belief, this thesis contains no material previously published or written by another person, except where due reference is made in the text of the thesis.

Ying-Lei Li August 2018

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[1] Li YL, Zhao XL, Raman Singh RK, Al-Saadi S. Experimental study on seawater and sea sand concrete filled GFRP and stainless steel tubular stub columns. Thin-Walled Structures. 2016;106:390-406.

[2] Li YL, Zhao XL, Raman Singh RK, Al-Saadi S. Tests on seawater and sea sand concrete-filled CFRP, BFRP and stainless steel tubular stub columns. Thin-Walled Structures. 2016;108:163-84.

[3] Li YL, Zhao XL, Singh Raman RK, Al-Saadi S. Thermal and mechanical properties of alkaliactivated slag paste, mortar and concrete utilising seawater and sea sand. Construction and Building Materials. 2018;159:704-24.

[4] Li YL, Teng JG, Zhao XL, Singh Raman RK. Theoretical model for seawater and sea sand concrete-filled circular FRP tubular stub columns under axial compression. Engineering Structures. 2018;160:71-84.

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[8] Li YL, Zhang YY, Nie RF, Zhao XL, Singh Raman RK, Al-Saadi S, et al. Tests of seawater and sea sand concrete (SWSSC) filled stainless steel (SS) and GFRP tubes. 11th International Conference on Advances in Steel and Concrete Composite Structures. Beijing, China2015. p. 335-41.

[9] Li YL, Zhao XL, Singh Raman RK, Al-Saadi S. Compression tests on seawater and sea sand concrete (SWSSC) filled stainless steel, CFRP and BFRP tubular stub columns. 8th International Conference on Fibre-Reinforced Polymer (FRP) Composites in Civil Engineering. Hong Kong, China2016. p. 270-6.

Thesis including published works declaration

I hereby declare that this thesis contains no material which has been accepted for the award of any other degree or diploma at any university or equivalent institution and that, to the best of my knowledge and belief, this thesis contains no material previously published or written by another person, except where due reference is made in the text of the thesis.

This thesis includes 5 original papers published in peer reviewed journals and 2 submitted publications. The core theme of the thesis is Structural Engineering. The ideas, development and writing up of all the papers in the thesis were the principal responsibility of myself, the student, working within the Department of Civil Engineering under the supervision of Prof. Xiao-Ling Zhao, Prof. R.K. Singh Raman and Dr. S. Al-Saadi.

(The inclusion of co-authors reflects the fact that the work came from active collaboration between researchers and acknowledges input into team-based research.)

Ch.	Publication Title	Status	Nature and % of student contribution	Co-author name(s) Nature and % of Co-author's contribution*	Co- author Monash student
2	Thermal and mechanical properties of alkali- activated slag paste, mortar and concrete utilising seawater and sea sand	Published	Developing ideas, establishing methodologies, experimental work, data analysis, write up and revision, 70%	 Prof. X.L. Zhao: Developing ideas, input into manuscript, revision, financial support, 15% Prof. R.K. Singh Raman Input into manuscript, Financial support, revision, 10% Dr. S. Al-Saadi: Revision, 5% 	No
3	Experimental study on seawater and sea sand concrete filled GFRP and stainless steel tubular stub columns	Published	Developing ideas, establishing methodologies, experimental work, data analysis, write up and revision, 70%	 Prof. X.L. Zhao: Developing ideas, input into manuscript, revision, financial support, 18% Prof. R.K. Singh Raman Revision, financial support, 7% Dr. S. Al-Saadi: Revision, 5% 	No
4	Tests on seawater and sea sand concrete-filled CFRP, BFRP and stainless steel tubular stub columns	Published	Developing ideas, establishing methodologies, experimental work, data analysis, write up and revision, 70%	 Prof. X.L. Zhao: Developing ideas, input into manuscript, revision, financial support, 18% Prof. R.K. Singh Raman Revision, financial support, 7% Dr. S. Al-Saadi: Revision, 5% 	No

In the case of Chapters 2-6 my contribution to the work involved the following:

5	Axial compression tests on seawater and sea sand concrete-filled double- skin stainless steel tubes	Published	Developing ideas, establishing methodologies, experimental work, data analysis, write up and revision, 70%	 Prof. X.L. Zhao: Developing ideas, input into manuscript, revision, financial support, 20% Prof. R.K. Singh Raman Revision, financial support, 5% Ms. Xiang Yu Experimental work, 5% 	Yes
6	Theoretical model for seawater and sea sand concrete-filled circular FRP tubular stub columns under axial compression	Published	Developing ideas, establishing methodologies, data analysis, write up and revision, 70%	 Prof. X.L. Zhao: Developing ideas, input into manuscript, revision, financial support, 15% Prof. J.G. Teng Input into manuscript, revision, 10% Prof. R.K. Singh Raman Revision, financial support, 5% 	No

I have renumbered sections of submitted or published papers in order to generate a consistent presentation within the thesis.

Student signature:

Date: 30/08/2018

The undersigned hereby certify that the above declaration correctly reflects the nature and extent of the student's and co-authors' contributions to this work. In instances where I am not the responsible author I have consulted with the responsible author to agree on the respective contributions of the authors.

Main Supervisor signature:

Date: 11/09/2018

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Introduction

1.1 Background

It is estimated that the global population will increase by 38%, from 6.9 billion in 2010 to 9.6 billion in 2050 (Kochhar 2014), which would increase the demand on natural resources (e.g., fresh water) and infrastructures. The world is now experiencing a serious water scarcity (IWMI 2006): one-thirds of global population (2 billion) live under conditions of severe water scarcity at least 1 month of the year and half a billion people face severe water scarcity all year round (Mekonnen and Hoekstra 2016). Any research that could reduce the fresh water consumption is of great significance to the world.

Based on a cradle-to-grave analysis (Miller et al. 2018), 16.6 billion tons of water, in the form of water withdrawal and water consumption as termed in Miller et al. (2018), were consumed in 2016 for concrete production, which equalled to 18% of global annual industrial water consumption. The extraction of river sand, which is the most common form of fine aggregate in concrete, impacts negatively on the river ecosystem. Furthermore, the emission of CO₂ during the production of Portland cement, which is from the combustion of fossil fuels and the conversion of calcium carbonate to oxide form, causes many environmental issues, such as greenhouse effect (Provis and Bernal 2014). In order to resolve the aforementioned problems, concrete containing seawater, sea sand and industry by-product (e.g. slag) is attracting many researchers' attention (Provis 2014).

In structural applications, as it is weak in tension, concrete is commonly used in accompany with steel, such as steel reinforced concrete structures and concrete-steel composite structures. However, the Chloride in seawater and sea sand can vastly impair the corrosion resistance of carbon steel. One solution is to replace carbon steel by corrosion resistant materials (e.g., fiber reinforced polymer composites and stainless steel). Fiber reinforced polymer (FRP) is increasingly being used in civil engineering due to the advantages of light weight, high strength, ease installation, low maintenance and high corrosion resistance (Bank 2006). For situations where metals have to be used, stainless steel (SS) is a desirable alternative to carbon steel as SS poses superior corrosion resistance due to its special chemical compositions. A hybrid construction form consisting of seawater and sea sand concrete (SWSSC), FRP and stainless steel was recently proposed and was attracting research interests (Teng 2014, Teng et al. 2011). Nevertheless, the short-term and long-term behaviours of this construction form have not been well addressed which may hinder its further application.

1.2 State of the art

Studies on seawater, sea sand and cement concrete (SSC) indicated that (Xiao et al. 2017): (1) the effect of seawater and sea sand on concrete workability is minimal and SSC exhibits a shorter initial and final setting time than ordinary concrete; (2) SSC has a higher early strength (i.e., 7-day strength)

whilst the long-term strength (after 28 days or more) is similar to that of ordinary concrete. In order to relieve the environmental issues raised by the production of ordinary Portland cement, alkaliactivated material, which is generally derived from industry by-products (e.g., alkali-activated slag), is being extensively studied as an alternative binder in concrete. However, to the author's best knowledge, concrete utilising seawater, sea sand and alkali-activated slag together (named as "SWSSC"), which is a more environment-friendly concrete, has not ever been proposed and investigated. In addition, the durability performance of SWSSC such as long-term behaviour in corrosive environments and performance in elevated temperature is lack of study.

The application of concrete-filled steel tubes (CFSTs) could be dated back to 1950s and CFST has become a mature and economic-efficient structural type in constructing high-rise buildings and bridges (Zhao et al. 2010). Due to the presence of Chloride in SWSSC, conventional carbon steel is not suitable and could be replaced by a more corrosive resistant material such as stainless steel (SS) or fiber reinforced polymer (FRP). Stainless steel has a rounded shape stress-strain curve without obvious yielding point, which contributes to a different structural behaviour of concrete-filled SS tubes in comparison to that of concrete-filled carbon steel tubes. Several studies have been conducted on concrete fully filled SS tubes but no study was conducted on concrete-filled double-skin SS tubes (Han et al. 2018). Concrete-filled double-skin tubes, which consists of two concentric steel tubes and sandwich concrete between them, have reduced self-weight and higher bending stiffness due to the existence of inner void.

It is well known that the strength and ductility of concrete could be significantly enhanced if being confined by fiber reinforced polymer (FRP) wrap. FRP wrap with fibers exclusively oriented in hoop direction has been widely applied in strengthening existing structures (Teng et al. 2002). Many stress-strain models for concrete confined by FRP wrap, in design-oriented form or analysis-oriented form (Teng and Lam 2004, Ozbakkaloglu et al. 2013), have been proposed and some of them were adopted in design standards (e.g., ACI 440.2R-08). FRP tube, which offers strength and stiffness in both longitudinal and hoop directions by orienting fibers in multiple directions, could be used as formwork for in-filled concrete and resist potential bending moment. However, the application of concrete-filled FRP tubes in new constructions is rare and it is lack of comprehensive studies on the structural behaviour of concrete-filled FRP tubes, let alone SWSSC-filled FRP tubes.

The durability of FRP has been extensively studied in recent decades and the external factors affecting the long-term behaviour of FRP include moisture, alkaline, acid, salt, temperature, stress, fatigue and ultraviolet radiation (Karbhari 2007). The degradation of FRP is a result of the deterioration of matrix, fiber and the interphase between them (Nkurunziza et al. 2005). Comparing to FRP composites, the

studies on the durability of concrete-filled FRP wraps/tubes are rather limited. The durability performance of seawater and sea sand concrete-filled FRP tubes in a marine environments has not ever been investigated.

1.3 Objectives

The present PhD thesis focuses on the behaviour of hybrid tubular sections utilising seawater and sea sand concrete (SWSSC), glass/carbon/basalt fiber reinforced polymer (G/C/B-FRP) and stainless steel (SS). Both the short-term structural behaviour and durability performance were investigated by the means of experimental study and theoretical analysis. The objectives of this thesis are:

(1) Behaviour of seawater and sea sand concrete

A mixture for alkali-activated slag-based seawater and sea sand concrete (SWSSC) was firstly developed through a "trial-and-error" method to achieve the desirable workability and strength. The mechanical properties of plain concrete at ambient temperature and elevated temperature were investigated. Slag paste, mortar and ordinary Portland cement concrete (using fresh water and river sand) were also tested for comparison purpose.

(2) Experimental study on short-term structural behaviour of seawater and sea sand concrete filled FRP and stainless steel tubes under axial compression

The major part of this research is to investigate the short-term structural behaviour of seawater and sea sand concrete (SWSSC) filled FRP and stainless steel circularly tubular stub columns under axial compression. The cross-section types are illustrated in Figure 1.1, which includes SWSSC fully filled FRP/SS tubes and SWSSC-filled double-skin tubes with different combinations of outer and inner tubes (i.e., SS outer-FRP inner or SS inner-FRP outer). Axial compression test was conducted on these SWSSC-filled specimens and corresponding unfilled hollow sections. The effects of key parameters, such as confinement factor, material strength, tube diameter, tube thickness and void ratio, on the column behaviours were discussed in depth.



Fig. 1.1. Cross-section types of SWSSC filled FRP and SS tubes

(3) Theoretical models for seawater and sea sand concrete filled FRP and stainless steel tubes

Theoretical model (i.e., analysis-oriented load-axial strain model) was proposed for SWSSC-filled FRP tubes under axial compression. In developing the theoretical model for SWSSC fully filled FRP tubes, the buckling of FRP tube under bi-axial stresses, the effects of biaxial stresses and Poisson effect on concrete dilation properties, and the contribution of FRP tube to resist axial load were properly considered. With regard to SWSSC-filled double-skin FRP tubes, the stress distribution in annular concrete is non-uniform, which is the major difference to concrete fully filled tubes. The concept of integration was adopted in this study to precisely account for the non-uniformly distributed stresses in concrete. As verified by the experimental results, the theoretical models could reasonably predict the load-axial strain curves of SWSSC-filled FRP tubes. In addition, design methods were proposed to estimate the ultimate capacity of SWSSC-filled SS tubes (both fully filled and double-skin tubes).

(4) Durability study on seawater and sea sand concrete filled FRP tubes in artificial sea water

In order to understand the durability performance of SWSSC-filled FRP tubes, accelerated degradation test (ADT) was conducted on the stub columns with the same dimensions to the columns for short-term behaviour experiments. The stub columns were immersed in 40 °C artificial seawater (3.5% NaCl solution) for different durations (i.e., 1-, 3-, and 6-month). After each duration, axial compression test was conducted to assess the effects of SWSSC and artificial seawater on the long-term behaviour of SWSSC-filled FRP tubes. In order to provide explanations for the capacity degradation of columns in material level, individual SWSSC and FRP rings in the same environment were also studied. Finally, design suggestions were proposed to account for the environment effects when applying SWSSC-filled FRP tubes in a marine environment.

1.4 Thesis outline

The research logic of this PhD thesis is presented in a flowchart shown in Figure 1.2. It is generally divided into two parts: short-term behaviour and durability at both material and member scales. Firstly, experiments were conducted on individual materials (SWSSC, SS and FRP) to obtain their mechanical properties, such as strengths and moduli. Axial compression test was then carried on SWSSC-filled FRP and SS tubular stub columns to understand their short-term structural behaviours. Based on the experimental study, theoretical models were developed for these columns. On the other hand, accelerated degradation test was conducted on both individual FRP and SWSSC-filled FRP tubes. The mechanical properties was adopted as the main indicators to represent their durability performances, which were compared to the unconditioned specimens. Finally, design suggestion was

proposed for the hybrid sections utilising SWSSC, FRP and SS, which could be applied in a marine environment.



Fig. 1.2. Research logic on hybrid sections utilising SWSSC, FRP and SS

This PhD thesis, which is publication-based, contains nine chapters (Figure 1.3). A brief introduction for each chapter are as following:

Chapter 1 Introduction

This chapter firstly provides a general introduction of the research backgrounds and significance, which could greatly relieve the environmental impacts by utilising environment-friendly material (seawater and sea sand concrete) and corrosion resistant materials (fiber reinforced polymer and stainless steel). The knowledge gaps are highlighted by reviewing existing studies and the originality and innovations of this research is further explained. The research objectives, including short-term and long-term behaviours of hybrid sections and research approaches (experiment and theoretical analysis) are introduced. The thesis structure is introduced in the end of this chapter.

Chapter 2 Thermal and mechanical properties of alkali-activated slag paste, mortar and concrete utilising seawater and sea sand

This chapter presents an experimental study on alkali-activated slag paste, mortar and concrete utilising seawater and sea sand at ambient temperature and exposed to elevated temperatures. Proper

mixtures were developed to achieve desirable compressive strength and workability. For heated specimens, both thermal and mechanical properties were investigated and compared to those of specimens at ambient temperature. In order to understand the mechanisms of the mechanical property degradation of concrete exposed to elevated temperature, X-ray CT scan and scanning electron microscopy (SEM) were utilised.

Chapter 3 Experimental study on seawater and sea sand concrete filled GFRP and stainless steel tubular stub columns

This chapter focuses on the short-term structural behaviour of seawater and sea sand concrete filled GFRP and stainless steel tubular stub columns. A total of 24 stub columns, including hollow tubes, fully filled tubes and double-skin tubes, were tested under axial compression. Tensile coupon test and split-disk test were conducted to obtain the material properties of stainless steel and GFRP. The stress-strain curves of the core concrete indicated that concrete strength and ductility were enhanced due to the confinement effect. The influence of tube diameter-to-thickness ratio, outer tube types and inner tube types on concrete confinement were discussed. Formulas were finally proposed to estimate the ultimate capacity of the stub columns.

Chapter 4 Tests on seawater and sea sand concrete-filled CFRP, BFRP and stainless steel tubular stub columns

Following the work in Chapter 3, this chapter extends the investigation to seawater and sea sand concrete-filled CFRP and BFRP tubes under axial compression. The methodology adopted herein is similar to that in Chapter 3. These two chapters forms the experimental work on SWSSC-filled GFRP, CFRP and BFRP tubes under axial compression, which is the solid foundation in developing the theoretical load-axial strain models in Chapters 6 and 7.

Chapter 5 Axial compression tests on seawater and sea sand concrete-filled double-skin stainless steel circular tubes

In order to fill the knowledge gap of SWSSC-filled double-skin stainless steel tubes, this chapter presents an experimental study on double-skin SS tubes under axial compression. SWSSC fully filled SS tubes and hollow SS tubes were also examined for comparison purpose. The effects of some key parameters, including confinement factor, void ratio and tube slenderness ratio, on the structural behaviour were clarified in this study. Formulas were proposed to estimate the load carrying capacity of SWSSC-filled SS circular tubes in compression, which formed part of the theoretical work on short-term behaviour.
Chapter 6 Theoretical model for seawater and sea sand concrete-filled circular FRP tubular stub columns under axial compression

FRP tube could offer substantial strength and stiffness in longitudinal direction, which leads to a different stress-strain relationship to that of concrete confined by FRP wrap. This chapter proposed an analysis-oriented load-axial strain model for SWSSC fully filled FRP tubes under axial compression. An existing dilation model was modified to properly account for the biaxial stresses and Poisson effect in FRP tube. The buckling of FRP tube was reasonably predicted by using an improved maximum strain failure criteria and the load carried by FRP tube was estimated by the proposed formulas. Finally, load-axial strain model was developed for SWSSC-filled FRP tubes by explicitly considering the properties of concrete, FRP tube and the interaction between them.

Chapter 7 Analysis-oriented load-strain model for concrete-filled double-skin circular FRP tubes under axial compression

This chapter is an extension of the theoretical model for SWSSC fully filled FRP tubes (Chapter 6) to double-skin tubes through substantial modifications. The stress in annular concrete of a double-skin tube is non-uniformly distributed and this phenomenon has not ever been implemented to theoretical models in existing studies. A constitutive model for non-uniformly confined concrete was firstly proposed based on existing researches on actively confined concrete. The dilation model developed in Chapter 6 was modified by introducing a term to consider the effect of void ratio. The annular concrete section was divided into multiple circular layers and the stress in each layer was determined. The stress carried by the entire section was calculated by integrating the layers together and an analysis-oriented load-axial strain model was finally proposed. The theoretical models proposed in this thesis showed reasonable accuracy as verified by the experimental results from the author and other researchers.

Chapter 8 Durability of seawater and sea sand concrete-filled FRP tubes in artificial seawater

This chapter presents some explorative work conducted by the author with regard to the durability performance of SWSSC-filled FRP tubes in an artificial seawater environment. The durability of SWSSC and FRP alone in the same environment was also examined for a better understanding of the capacity degradation mechanism of stub columns. The specimens were immersed in 40 °C saltwater for different durations (i.e., 1-, 3-, and 6-month) and mechanical tests were conducted after each duration. Load-axial strain curves and ultimate capacity of SWSSC-filled FRP tubes were predicted by using the previous proposed model in this thesis with proper modification to account for the

environmental effects. Design suggestions were given in the end of this chapter for SWSSC-filled FRP tubes in a marine environment.

Chapter 9 Conclusions and future work

This chapter summarizes the conclusions based on the study in this thesis and gives suggestions for future work.

	Ch. 1	Introduction
SWSSC	Ch. 2	Thermal and mechanical properties of alkali-activated slag paste, mortar and concrete utilising seawater and sea sand
Experimental study on short-term structural behaviour	Ch. 3	Experimental study on seawater and sea sand concrete filled GFRP and stainless steel tubular stub columns
	Ch. 4	Tests on seawater and sea sand concrete-filled CFRP, BFRP and stainless steel tubular stub columns
	Ch. 5	Axial compression tests on seawater and sea sand concrete-filled double-skin stainless steel circular tubes
Theoretical model	Ch. 6	Theoretical model for seawater and sea sand concrete-filled circular FRP tubular stub columns under axial compression
	Ch. 7	Analysis-oriented load-strain model for concrete-filled double-skin circular FRP tubes under axial compression
Durability	Ch. 8	Durability of seawater and sea sand concrete-filled FRP tubes in artificial seawater
	Ch. 9	Conclusions and future work

Fig. 1.3. Thesis structure

It should be pointed out that this PhD thesis follows the format of "Thesis including published works" set by Monash University. Each chapter (from Chapter 2 to Chapter 8) has its own abstract, keywords, nomenclature, introduction, acknowledgement and references. Inevitably, some repetitions exist. There is no Literature Review chapter as in traditional PhD thesis because each chapter has already included relevant literature review.

References

ACI 440.2R-08 (2008), Guide for the design and construction of externally bonded FRP systems for strengthening concrete structures, American Concrete Institute, Farmington Hills, MI.

Bank, L.C. (2006), Composites for construction: Structural design with FRP materials, John Wiley & Sons, Hoboken, New Jersey.

Han, L.-H., Xu, C.-Y., and Tao, Z. (2018), Performance of concrete filled stainless steel tubular (CFSST) columns and joints: Summary of recent research, Journal of Constructional Steel Research. (in press)

IWMI (2006), Insights from the comprehensive assessment of water management in agriculture, International Water Management Institute (IWMI), Colombo, Sri Lanka.

Karbhari, V.M. (2007), Durability of composites for civil structural applications, Woodhead Publishing Limited, Cambridge, England.

Kochhar, R. (2014), Projections for the global population in 2050, Pew Research Center.

Lawrence, D. (2015), Experimental evaluation of fiber reinforced polymer piles in load-bearing application, The University of Maine.

Mekonnen, M.M. and Hoekstra, A.Y. (2016), Four billion people facing severe water scarcity, Science Advances, 2, e1500323.

Miller, S.A., Horvath, A., and Monteiro, P.J. (2018), Impacts of booming concrete production on water resources worldwide, Nature Sustainability, 1, 69-76.

Nkurunziza, G., Debaiky, A., Cousin, P. and Benmokrane, B. (2005), Durability of GFRP bars: A critical review of the literature, Progress in Structural Engineering and Materials, 7, 194-209.

Ozbakkaloglu, T., Lim, J.C. and Vincent, T. (2013), FRP-confined concrete in circular sections: Review and assessment of stress-strain models, Engineering Structures, 49, 1068-88.

Provis, J.L. and Bernal, S.A. (2014), Geopolymers and related alkali-activated materials, Annual Review of Materials Research, 44, 299-327.

Provis, J.L. (2014). Geopolymers and other alkali activated materials: why, how, and what?, Materials and Structures. 47. 11-25.

Teng, J., Chen, J.-F., Smith, S.T. and Lam, L. (2002), FRP: Strengthened RC structures, Frontiers in Physics, 266.

Teng, J.G. and Lam, L. (2004), Behavior and modeling of fiber reinforced polymer-confined concrete, Journal of Structural Engineering, 130, 1713-23.

Teng, J.G., Yu, T., Dai, J.G. and Chen, G.M. (2011), FRP composites in new construction: current status and opportunities, in: Proceedings of 7th National Conference on FRP Composites in Infrastructure (Supplementary Issue of Industrial Construction), keynote presentation.

Teng, J.G. (2014), Performance enhancement of structures through the use of fibre-reinforced polymer (FRP) composites, in: Proceedings of 23rd Australasian Conference on the Mechanics of Structures and Materials (ACMSM23), Lismore, Australia, keynote presentation.

Xiao, J., Qiang, C., Nanni, A. and Zhang, K. (2017), Use of sea-sand and seawater in concrete construction: Current status and future opportunities, Construction and Building Materials, 155, 1101-11.

Zhao, X.L., Han, L.H., and Lu, H. (2010), Concrete-filled tubular members and connections, CRC Press, London.

Chapter 2

Thermal and mechanical properties of alkali-activated slag paste, mortar and concrete utilising seawater and sea sand

Abstract

This chapter presents an experimental study on the thermal properties of alkali-activated slag paste, mortar and concrete utilising seawater and sea sand exposed to elevated temperature. The thermal properties of paste and concrete utilising cement, fresh water and river sand were also investigated for comparison purpose. The samples were heated to different target temperatures up to 1000 °C at a heating rate of 5 °C/min, and tested both under hot and cooled conditions. The thermal properties, including temperature gradient, visual observation, mass loss, thermal strain and mechanical properties (i.e. strength, Young's modulus and stress-strain curve) were investigated. X-ray CT scanning and scanning electron microscopy (SEM) were conducted to understand the macro/microscopic changes of the paste and concrete in response to heating. Degradation in mechanical properties of slag paste is attributed mainly to the cracks induced by temperature gradient, pore pressure and phase change. The degradation of concrete is caused by thermal mismatch between paste matrix and aggregates regardless of the use of cement or slag, freshwater or sea water, and river sand or sea sand.

Keywords

Thermal properties, alkali-activated concrete, seawater and sea sand concrete (SWSSC), elevated temperature

2.1 Introduction

As the most widely used construction material, concrete has the advantages of low cost, easy maintenance, flexible workability and good fire resistance. The binding phase which provides strength to a concrete is usually based on ordinary Portland cement (OPC). The rapidly increasing demand of concrete generates significant environmental issues. About 0.8 tonnes of CO₂ is emitted to produce 1 tonne of Portland cement, i.e., from the combustion of fossil fuels and the conversion of calcium carbonate to oxide (Provis and Bernal 2014). Besides, large quantities of fresh water and river sand are also consumed in the concrete industry, which exacerbate the resource shortage. In recent decades, an alternative cement-like binder called 'alkali-activated material (AAM)', which is formed by the interaction between aluminosilicate precursors (which are generally industrial by-products) and alkaline activators, was proposed and extensively investigated (Ding et al. 2016; Provis 2014; Davidovits 2015; Shi et al. 2006). AAMs are also called geopolymers in some literatures. To avoid the high consumption of fresh water and river sand in concrete industry, seawater and sea sand have attracted the researchers' attention as potential candidates for construction industry (Teng 2014; Teng et al. 2011; Katano et al. 2013; Mohammed et al. 2004; Nishida et al. 2015; Xiao et al. 2017).

Research on hybrid construction system utilizing seawater, sea sand and fiber reinforced polymer is being carried out at Monash University in collaboration with The Hong Kong Polytechnic University and Southeast University, China. Earlier work out of this exercise concerns to the structural behaviour of FRP confined seawater and sea sand concrete (Li et al. 2016a, b) and the durability of FRP bars in seawater and sea sand concrete environment (Wang et al. 2017a, b).

The most common precursors used in AAM are ground granulated blast furnace slag (GGBFS), fly ash and metakaolin, among which slag has much higher calcium content. The alkaline activation requires the addition of activator (e.g. sodium hydroxide, sodium silicate and potassium hydroxide in solution or dry forms), which is the major difference to Portland cement. Generally, the reaction mechanism involved in alkaline activation includes dissolution, rearrangement, condensation and resolidification (Provis 2014; Davidovits 2015; Singh et al. 2015; Provis and Van-Deventer 2009). Calcium aluminium silicate hydrate (C-A-S-H) gel with a disordered tobermorite-like structure is the main reaction product in alkali-activated slag (Provis and Bernal 2014).

The behaviour of concrete at elevated temperatures or in fire is one of the major concerns when considering durability of concrete structures. In comparison to steel and timber, concrete has the advantages of low thermal diffusivity and incombustibility in fire. However, the concrete at elevated temperature exhibits mechanical properties deterioration due to the physicochemical changes in paste and aggregate and the thermal incompatibility between paste and aggregate (Khoury 1992; Khoury 2000; Ma et al. 2015). The studies on Portland cement concrete (Khoury 2000; Hager 2013; Neville 2011) find the decomposition temperatures of calcium hydroxide and calcium carbonate to be 300-500 °C and 700-1000 °C respectively, and their melting starts at 1000-1300 °C. Other changes like crystallisation and phase changes have been observed in alkali-activated materials exposed to elevated temperature (Duxson et al. 2007). The morphology of the geopolymer paste can be altered by manipulating the schedules of sintering, crystallisation and melting, whilst dehydration and densification can affect the size and distribution of the pore structure (Rashad et al. 2016).

Different properties of geopolymers and geopolymer composites (mortar and concrete) exposed to elevated temperature have been investigated by several research groups, e.g., the effects of ductility and transit creep on residual strength (Pan and Sanjayan 2010; Pan et al. 2014; Pan et al. 2009), the residual strength of alkali-activated slag concrete (Guerrieri et al. 2009), the spalling behaviour of geopolymer and Portland cement concrete in simulated fire (Zhao et al. 2011), the effects of specimen size, aggregate size, aggregate type, precursors, and activators on the thermal properties (Kong and Sanjayan 2008; Kong and Sanjayan 2010; Kong et al. 2007). Also, the thermal properties of alkali-activated slag paste and mortar are reported (Rashad et al. 2016; Khater 2014; Rovnanik et al. 2013; Wang et al. 2014; Zuda et al. 2006). Recently, Aslani (2015) summarized the reported researches on

the mechanical properties of geopolymers exposed to elevated temperature, and proposed constitutive models for strength and modulus. Besides the mechanical properties, morphology, phase changes and pore structures of geopolymers and geopolymer composites are investigated by means of scanning electron microscope (SEM), X-ray diffraction analysis (XRD), thermogravimetric and derivative thermogravimetric analysis (TGA/DTG), Differential thermal analysis (DTA), FTIR spectroscopy, and mercury intrusion porosimetry (Povnanik et al. 2013; Wang et al. 2014; Abdulkareem et al. 2014), which are helpful explaining the strength change. In general, the strength change is a resultant of phase transformation, thermal incompatibility and pore structure effects.

Up to now, no studies have been conducted on the behaviour of alkali-activated concrete using seawater and sea sand exposed to elevated temperature or fire (Xiao et al. 2017), which may limit further application of this 'environment-friendly' concrete. The research in this chapter aims to fill up this knowledge gap. This chapter focuses on the thermal properties of alkali-activated slag paste, mortar and concrete that was prepared using seawater and sea sand, and then exposed to elevated temperature. The ordinary Portland cement paste and concrete were also investigated for comparison purpose. The temperature gradient, mass loss, thermal strain and mechanical properties, such as residual strength, hot strength, Young's modulus, stress-strain curve, were studied for paste, mortar and concrete. The X-ray CT scanning and SEM are conducted to characterize the macro/microstructures of paste and concrete after high temperature exposure.

2.2 Experimental investigation

2.2.1 Materials

The ground granulated blast furnace slag (GGBFS, called 'slag' in short throughout this chapter), which is a by-product of steel-making, was adopted as cementitious material in this research. The chemical composition of slag was determined by X-ray fluorescence (XRF) and is listed in Table 2.1. Its chemical composition is consistent with most of the existing researches (Shi 2006), with Si-to-Al ratio of 2.19. The calcium oxide (CaO) content in slag is much higher compared to that in fly ash or metakaolin and less in Portland cement.

Table 2.1. Chemical composition of slag, sodium meta-silicate and sea sand (wt %)

Tuble 2.1. Chemiear composition of shag, socialit meta sincate and sea sand (we vo)													
Compound	SiO ₂	Al_2O_3	Fe_2O_3	CaO	MgO	Na ₂ O	K_2O	MnO	TiO_2	P_2O_5	Cl	SO_4	PO_4
Slag	34.8	13.5	0.61	41.3	5.19	0.19	0.33	0.28	0.53	0.03	-	-	-
Sodium meta-silicate	46.6	0.06	0.03	0.04	0.01	35.8	0.2	-	-	-	-	-	-
Sea sand	96.5	0.21	0.43	1.3	0.06	0.07	0.05	-	0.04	-	0.13	0.09	0.01

The powdered sodium meta-silicate with modulus ratio of 1.35 was used as an activator in this research. The chemical composition of sodium meta-silicate is shown in Table 2.1. Following the

suggestions made by Collins (1999) and Cheng and Sarkar (1994) on how to improve the workability, 1 % (weight percentage of slag) hydrated lime slurry, which consisted of 1/4 lime and 3/4 water by weight, was added to the mixture. The slump of SWSSC (with hydrated lime slurry, 14 mm coarse aggregate) is about 140 mm.

The sea sand and seawater were obtained in the coastal beach near Melbourne. Sieve analysis was carried out according to AS 1141.11.1 (2009) to determine the particle size distribution (PSD) of sea sand. The PSD of sea sand is compared in Fig. 2.1 with the river sand used in this research. 51.7% and 13.3% of the sea sand could respectively pass the sieves with the apertures of 0.6 mm and 0.3 mm, whilst the corresponding percentages for river sand are 79.4% and 36.8% respectively. The fineness modulus of sea sand is 2.39 and that of river sand is 1.86, indicating the sea sand is slightly coarser than the river sand. The chemical composition of sea sand is listed in Table 1, in which the weight percentage of Cl^- ions is 0.13%. The chemical composition of seawater used in this research is shown in Table 2.2, where the concentration of NaCl is about 30 g/l.



Fig. 2.1. Particle size distribution curve of sea sand and river sand

Table 2.2. Chemical composition of seawater										
Ion	Ca	Mg	Na	K	Cl	SO_4				
mg/L	439	1430	11940	622	20700	3420				

Two kinds of coarse aggregate were adopted in this study: basalt with maximum size of 7 mm and basalt with maximum size of 14 mm. The ordinary Portland cement (OPC) is a general purpose cement produced in Australia. The tap water provided in the laboratory is adopted as fresh water.

2.2.2 Sample preparation

A total of eight mixtures, including three kinds of paste (SWP – slag paste using seawater; FWP – slag paste using freshwater; OPCP – ordinary Portland cement paste using freshwater), one kind of mortar (SWSSM – slag based seawater and sea sand mortar), and four kinds of concrete (SWSSC – slag based seawater and sea sand concrete; FWRSC – slag based freshwater and river sand concrete;

OPCC – ordinary Portland cement based freshwater and river sand concrete; SWSSC2 – slag based seawater and sea sand concrete utilising coarse aggregate with larger size (~14 mm)), were investigated in this chapter. The water-to-binder (slag or OPC) ratio is fixed as 0.53 and the fine-to-coarse aggregate ratio is 0.73. The mix proportions are summarised in Table 2.3.

Constituents (kg/m ³)	SWP	FWP	OPCP	SWSSM	SWSSC	FWRSC	OPCC	SWSSC2
Slag	360	360		360	360	360		360
Cement (OPC)			360				360	
Seawater (SW)	190			190	190			190
Freshwater (FW)		190	190			190	190	
Sea sand (SS)				830	830			830
River sand (RS)						830	830	
Coarse aggregate (~7 mm)					1130	1130	1130	
Coarse aggregate (~14 mm)								1130
Sodium meta-silicate	38.4	38.4		38.4	38.4	38.4		38.4
Hydrated lime slurry	14.4	14.4		14.4	14.4	14.4		14.4

The activator (i.e. sodium meta-silicate) was pre-blended with slag in dry state before mixing. The mixing sequence for mortar and concrete is: premixing the aggregates with 1/3 water for 30 s, adding binder (slag pre-blended with activator or OPC), mixing for 2 min during which adding the rest of water and lime slurry simultaneously (if applicable), resting for 2 min, and mixing for another 2 min. The step of premixing aggregates was skipped when mixing the paste. The mixtures were poured into the moulds in three equal layers, with mechanical vibration for each layer for about 30 s to achieve compaction in each layer. All the samples were cured under polyethylene sheet for 1 day at ambient temperature before demoulding. After that, the samples were sealed by plastic films and stored at ambient temperature for about 1 month until testing.

The sample is cylindrical with the dimensions of 30 mm dimeter and 60 mm height for all the mixtures except SWSSC2, whose sample dimensions are 50 mm diameter and 100 mm height. The dimensions of the samples satisfy the requirement in ASTM C192/C192M (2007), which states that the diameter of a cylindrical specimen shall be at least three times the nominal maximum size of the coarse aggregate. In order to guaranty the ends of samples are flat and parallel, the samples were ground by a grinding machine before heating or compression test.

2.2.3 Heating regime

The samples were heated to the target temperatures (i.e. 100, 200, 400, 600, 800 or 1000 °C) at a heating rate of 5 °C/min from room temperature in a muffle furnace. Once the target temperature was reached, it was then held for 30 min (except SWSSC2 that is held for 90 min) to ensure homogeneity of temperature across the sample. After the duration, the furnace was switched off and the sample

was naturally cooled down to room temperature within the furnace. The heating schedule is shown in Fig. 2.2. In this research, some samples (i.e., SWSSC, FWRSC and OPCC) were tested in hot condition. These samples were heated in a different furnace installed on the compression test machine at the same heating rate as well as the period of temperature homogenisation.



Fig. 2.2. Heating schedule (schematic view)

2.2.4 Test methods

2.2.4.1 Temperature gradient

In order to monitor the temperature change, two thermocouples were used to measure the temperature of the furnace air and the temperature at the centre of the sample. The temperature gradient is the difference of the temperature in air and at centre divided by the radius of the cylinder. In the current study, only the temperature gradients of slag concretes with different dimensions (i.e. $30 \text{ mm} \times 60 \text{ mm}$ for SWSSC and $50 \text{ mm} \times 100 \text{ mm}$ for SWSSC2) were measured.

2.2.4.2 Mass loss

The mass loss of the samples were determined by weighing the samples before and after heating. Mass loss test were carried out in triplicate for each case. In order to understand the effect of elevated temperatures on mass loss of samples, mass losses at a few specific temperatures were measured.

2.2.4.3 Thermal strain

Thermal strain is the expansion/contraction of the sample during heating, which is measured by dilatometer testing. Thermal strain is of special interest as the expansion or contraction during heating causes both internal and external stresses, which potentially weakens or damages the structure. In this chapter, the thermal strain is defined as the length difference of the sample before and during heating divided by the initial length (i.e., before heating).

The test setup for thermal strain measurement is shown in Fig. 2.3, in which two loading rods touch the sample ends with a negligible prestress (0.1 MPa) during the heating process. The compression machine was set as load control and the negligible load was held consistent by moving the upper

loading rod up and down automatically. The position of the loading rod was recorded, which is the total expansion/contraction of the sample and the loading rods within the furnace. Similar measurement was conducted on loading rods (without sample) to obtain the expansion of loading rods during heating. The expansion/contraction of the sample is the total value subtracted by the contribution of the loading rods within the furnace.



Fig. 2.3. Test setup for thermal strain measurement

Residual thermal strain was also measured, which was the length difference of the sample before and after heating (cooling down to room temperature) divided by the initial length before heating. Throughout this chapter, the positive thermal strain means the expansion of sample whilst the negative strain represents contraction.

2.2.4.4 Compressive test

Axial compressive test was conducted on samples with and without temperature exposure. The loading rate for samples with dimension of 30×60 mm is 0.15 mm/min with displacement control, whilst that for samples with dimension of 50×100 mm is 0.20 mm/min. For samples with greatly reduced stiffness (e.g. samples exposed to ≥ 800 °C), the loading rate was increased to make sure the compressive test was conducted at a reasonable stress rate.

The compressive test is classified into three groups: reference test on samples without temperature exposure (denoted as '23 °C'), residual strength test on cooled samples after exposure, and hot strength test on samples in hot condition. The laser extensometer was used to obtain the stress-strain curves of samples without or after temperature exposure. In order to make a direct comparison between samples tested at room temperature and hot condition, the Young's modulus was calculated based on the displacement readings recorded by the test machine. The absolute value of the Young's modulus for unexposed samples was determined from two vertically placed strain gauges.

The compressive test was finished within 5 days for each mixture to eliminate the influence of strength development along curing age. Three identical samples were tested for each case and the average test results were reported and discussed in this chapter.

2.2.4.5 X-ray CT scanning

High-resolution X-ray Computed Tomography (CT) scanning method was adopted in this research to obtain the non-destructive three dimensional images of samples before and after elevated temperature exposure. This technique enables characterisation, without damaging the samples, of changes in macrostructure (e.g. cracks) that can be caused by temperature exposure. Post-processing of images was carried out using software AVIZO (v9.0.1).

The samples selected for X-ray CT scanning include slag paste using seawater (SWP), slag concrete using seawater and sea sand (SWSSC) and ordinary Portland cement concrete using freshwater and river sand (OPCC). Both the unexposed samples and samples after 600 °C exposure were examined. The dimensions of paste samples are 30 mm diameter and 30 mm height, whilst that of concrete sample is 30 mm diameter and 10 mm height. The scanning resolution is 30 microns, which means one pixel in the image represents 0.03 mm.

2.2.4.6 Scanning electron microscopy (SEM)

In order to understand the microstructure (e.g. morphology) of slag paste after elevated temperature exposure, scanning electron microscopy (SEM) was conducted on sample fragments using a JEOL JSM-7001F. Only the slag paste using seawater (SWP) without exposure and with exposure of 200 °C, 600 °C and 1000 °C were examined by SEM.

2.3. Results and discussions

2.3.1 Temperature gradient

The temperature-time curves of concrete cylinders with dimensions of 30×60 mm (SWSSC) and 50×100 mm (SWSSC2) are shown in Fig. 2.4a and b. It is assumed that the furnace air temperature is the same as the temperature at the sample surface. The temperature difference (noted as 'Furnace air – Centre') divided by the distance from centre to surface is regarded as the temperature gradient, which is shown in Fig. 2.4c.



Fig. 2.4. Measured temperature development in concrete

As expected, the temperature rise or drop at the centre was slower than that of the furnace air during the heating or cooling stage. It is shown that the soaking time for target temperature is long enough for the entire sample to attain homogeneous temperature. The small difference between surface and centre temperatures indicates that it is reasonable to use the furnace air temperature to represent sample's exposure temperature when plotting the thermal strain-temperature curves in Section 2.3.4. As shown in Fig. 2.4c, the maximum temperature gradient occurs in the range of 100 °C – 200 °C (temperature at centre), at which most of the water loss happens. The conversion of water into steam may retard the temperature increase and lead to the maximum temperature gradient. In general, the samples with dimension of 50 × 100 mm have slightly higher temperature gradient, which can be attributed to their larger diameters.

Snell et al. (2017) found that the thermal conductivity of geopolymer paste is much lower than that of cement paste. With the decrease of paste percentage in a concrete mixture, the conductivity increases for both geopolymer concrete and cement concrete. The conductivities of the two concretes become similar when paste (with regardless of slag or cement) percentage is about 15%, which is same to the mixture used in this study. The similar conductivities of slag concrete and cement concrete

are caused by the fact that the aggregates dominate the thermal conductivity due to its high volume occupation. Therefore, it is expected that paste has higher temperature gradient than concrete and the temperature gradient of slag paste is higher than that of Portland cement paste.

2.3.2 Visual observation

23 °C

2.3.2.1 Cross-section images

The cross-section images of paste, mortar and concrete after exposure to elevated temperature are shown in Fig. 2.5. Before heating, all the cylindrical samples were ground to reveal the aggregates.

Considerable cracks were observed in slag paste (SWP) upon heating at temperatures ≥ 200 °C. These cracks are caused by the pressure of evaporating water and temperature gradient. The cracks observed for mortar and concrete is not obvious in the cross-section images. Only small cracks are found along the interfaces of the coarse aggregate and paste matrix at 800 °C and 1000 °C. The cracks around coarse aggregate in slag concrete are more obvious than those in cement concrete (OPCC) as the thermal incompatibility between slag paste and coarse aggregate is severer than that between cement paste and coarse aggregate (as discussed in Section 2.3.4).



200 °C400 °C600 °C800 °C1000 °C(c) SWSSC (slag concrete using seawater and sea sand)



Fig. 2.5 also shows obvious colour change in paste, mortar and concrete after temperature exposure. Colour is an important indicator of the temperature the sample experienced during a fire. Some relevant studies (Arioz 2007; Hager 2014; Short et al. 2001) have been conducted on cement concrete, but reports on the colour change of slag concrete are rather limited.

As shown in Fig. 2.5a, the slag paste displays a dull blue-green colour before exposure, which is similar to the colour reported for slag/cement blends (ACI233R-03 (2011)). With the increase of temperature, the colour of slag paste turns from dull blue-green ($\leq 400 \,^{\circ}$ C) to brown (600 $^{\circ}$ C), grey (800 $^{\circ}$ C) and white (1000 $^{\circ}$ C). It is generally agreed (Hager 2014; Annerel and Taerwe 2009) that ordinary Portland cement paste turns whitish-grey at the temperature around 600 $^{\circ}$ C due to decomposition of portlandite into CaO. The mortar prepared using seawater and sea sand (SWSSM) turns red at 600 $^{\circ}$ C probably due to formation of new iron compounds. Fig. 2.5c-f indicate the colour of the coarse aggregate to turn from blue ($\leq 400 \,^{\circ}$ C) to red (600-1000 $^{\circ}$ C). The colour change of aggregate depends largely on the aggregate type and mineral compositions (Hager 2014; Annerel and Taerwe 2009). The colour change of concrete is a combinational results of that of paste, fine aggregate and coarse aggregate, in which aggregate plays a dominant role due to its much higher volume occupation than paste matrix.

2.3.2.2 Surface images

In this research, spalling was observed exclusively in cement paste (OPCP) heated to 600 °C and 800 °C (Fig. 2.6). However, no spalling occurred in other paste, mortar or concrete samples. Spalling is generally attributed to the vapour pressure in pores and thermal stresses (Khoury 2000; Ma et al. 2015; Hager 2013). Based on this research, the slag paste shows superior spalling resistance than Portland cement paste. Sarker et al. (2014) and Zhao and Sanjayan (2011) reported that geopolymer concrete exhibits a better spalling resistance than cement concrete. However, it is difficult to compare the spalling resistance of slag concrete and cement concrete in this study as spalling did not occurred in these two concretes.



(a) 600 °C (b) 800 °C Fig. 2.6. Spalling of OPCP exposed to 600 °C and 800 °C



(a) SWP

(b) FWP Fig. 2.7. Surface of pastes after 1000 °C exposure



(e) SWSSC2 (a) SWSSM (d) OPCC (b) SWSSC (c) FWRSC (f) enlarged image of (e) Fig. 2.8. Surface of mortar and concrete samples after 800 °C exposure.

Pronounced surface cracks were observed for paste samples at ≥ 600 °C, whereas such cracks in mortar and concrete samples did not eventuate until 800 °C. The typical surface crack patterns are illustrated in Fig. 2.7 and 2.8. Only a small number of large scale cracks appeared on the paste sample surface (Fig. 2.7) whereas minor cracks were densely distributed on the mortar and concrete sample surface (Fig. 2.8). The CT scan images of paste samples (discussed in Section 2. 4.1) have indicated that the cracks propagate from the core to the surface, which suggests the temperature gradient to be the cause. On the other hand, the cracks on mortar and concrete surfaces are caused by the fact that the shrinkage of paste matrix in the surface layer is restrained by the underneath aggregates and tensile stress is formed there.

2.3.3 Mass loss

The mass loss of all the mixtures after elevated temperature exposure is summarized in Fig. 2.9, which indicates a gradual mass loss with the increase of temperature. The mass loss is generally caused by the loss of free water or physically and chemically bonded water (Mehta and Monteiro 2006). The maximum mass loss is very close to the water content in each mixture, indicating that almost all the water can be lost during heating to very high temperature (e.g. 1000 °C in the present study).



As shown in Fig. 2.9a, the mass loss trend of slag paste (SWP and FWP) is different from that of Portland cement paste (OPCP). Slag paste has a higher percentage of mass loss than cement paste at the temperature range of 200 - 600 °C, which is attributed to the different mechanisms for hydration of geopolymer and cement paste (Davidovits 2015; Taylor 1997). More free water exists in geopolymer acting as a solvent, whereas much greater fraction of water is physically and chemically bonded in the case of cement paste. Such water is released at higher temperatures due to the

dehydration of C-S-H gel and decomposition of portlandite. The mass loss curve of slag paste using seawater (SWP) is very similar to that of FWP, with only a minor variation at 100 °C. The simplest explanation could be the presence of NaCl since the presence of dissolved salt is well-known to elevate the evaporation temperature (i.e., retardation of water evaporation during heating).

The mass loss curves of slag concretes (SWSSC, FWRSC and SWSSC2) and ordinary Portland cement concrete (OPCC) are similar (Fig. 2.9b). Most of the mass loss occurred at temperatures \leq 400 °C since the mass loss is primarily due to the water loss of paste. The maximum mass losses are in agreement with the water contents of the concretes. The weight percentage of paste matrix in concrete is only about 21%, therefore the different mass loss behaviour of pastes does not affect the mass loss of concretes obviously. Furthermore, the presence of seawater and sea sand in the concrete has an insignificant effect on its mass loss.

The mass loss curves of paste, mortar and concrete using seawater and sea sand are plotted in Fig. 2.9c. It is obvious that the higher water content leads to higher mass loss. As expected, the influence of aggregate size and sample dimension (SWSSC vs. SWSSC2) on mass loss is negligible.

2.3.4 Thermal strain

2.3.4.1 Thermal strain in hot condition

The thermal strain - temperature curves of slag paste with seawater (SWP) and cement paste with freshwater (OPCP) are shown in Fig. 2.10a. Both the SWP and OPCP shrank at elevated temperatures.



Fig. 2.10. Thermal strain of paste and concrete

The nature of the curve for SWP is largely similar to the reported results for alkali-activated materials (fly ash: Kong and Sanjavan 2010; Rickard et al. 2012, metakaolinite: Duxson et al. 2007; Rahier et al. 1997), but it is different from the curve reported by Guerrieri et al. (2009) for alkali-activated slag paste. The thermal strain curves for SWP during heating up to 750 °C in this study can be divided into four stages. The dimensions of SWP stay stable until heating up to 150 °C due to the balance of the contrary effects of the expansion of solid upon heating and the shrinkage caused by loss of water. It should be mentioned that the temperature data used in developing Fig. 2.10 is the furnace air temperature. As shown in Fig. 2.4a, there is about a 50 °C lag for the temperature in sample centre when furnace air temperature ranges from 50 °C to 200 °C. The second stage in the thermal strain curve for SWP is a rapid contraction between 150 °C and 400 °C. As shown in Fig. 2.9a, most of the mass loss occurs in this region. This shrinkage is mainly caused by the loss of water (Duxson et al. 2007) causing the shrinkage of pores and dehydroxylation in geopolymers (Rahier et al. 1997) which leads to the shrinkage of geopolymer gel. With completion of water loss assisted shrinkage, the thermal strain stays almost unchanged in the temperature range of 400 - 600 °C, which marks the third stage. However, considerable shrinkage occurs again in the temperature range of 600 - 750 °C, which may be attributed by softening and viscous flow of the aluminosilicate material at elevated temperatures that is reported to cause sintering and subsequent densification (Rickard et al. 2012). This rapid shrinkage is also an indication of the glass transition temperature (T_g) (Rahier et al. 1997).

The thermal strain regimes for Portland cement paste (OPCP) are in agreement with previous study (Cruz and Gillen 1980). These regimes include a dimensionally stable (or slightly expansion) region (<150 °C), linear shrinkage region (150 - 700 °C, due to water loss), a short expansion region (700 - 750 °C) and a region of further shrinkage (>800 °C, that was not identified in this research).

It is generally agreed that the loss of free water does not cause shrinkage, but the loss of water held by capillary tension, physical or chemical bonding (called 'capillary water in small voids', 'adsorbed water', 'interlayer water', 'chemically combined water' in Mehta and Monteiro 2006) will cause shrinkage. The alkali-activated slag paste exhibits higher shrinkage than ordinary Portland cement paste probably due to their different pore structure as well as due to difference in shrinkage behaviour of C-A-S-H gel of geopolymer and C-S-H gel of hydrated Portland cement.

During the cooling, slight expansion is observed from 750 °C to 400 °C, and further shrinkage occurs at temperatures below 400 °C due to the shrinkage of solid upon cooling. Most of the shrinkage is irreversible as the pore structure of paste has been altered mainly due to the loss of water. Because of different hydration mechanism, the thermal strains of cement paste and slag paste are different. The shrinkage of slag paste is severer than that of cement paste during heating up to 750 °C.

The thermal strain curves of slag concrete utilizing seawater and sea sand (SWSSC) and ordinary Portland cement concrete (OPCC) are plotted in Fig. 2.10b, in which a comparison with the thermal expansion of coarse aggregate (basalt) reported by Kong and Sanjayan (2008) is also presented. Even though the paste shrinks during heating, both SWSSC and OPCC expands and the trends are similar. As aggregate occupies about 80% of the concrete weight, the expansion of aggregate dominates the thermal strain behaviour of concrete. In this research, the SWSSC undergoes greater expansion than OPCC, even though slag paste shrinks more than cement paste, probably due to the different thermal expansion behaviour of sea sand from river sand and the swelling of the impurity in the sea sand, such as shell debris. As shown in Fig. 10a, the thermal incompatibility between basalt and slag paste is severer than cement paste. Larger amount of cracks in SWSSC than in OPCC (confirmation described in Section 2.4.1) may also contribute to a higher expansion of SWSSC. It is found that most of the thermal strain of concrete is reversible because the expansion of basalt is reversible.

Based on Fig. 2.10 (that presents thermal strain behaviour up to 750 °C) and a comparison to existing research on paste and concrete using freshwater, it can be concluded that the seawater does not have any obvious additional effect on the thermal strain of slag paste or concrete, but the sea sand and its impurity could cause an extra expansion of concrete.

2.3.4.2 Residual thermal strain

The residual thermal strains of paste, mortar and concrete after elevated temperature exposure are shown in Fig. 2.11. The slag pastes (SWP and FWP) exhibit larger residual thermal strain than OPCP, which is in agreement with the thermal strain curves in Fig. 2.10a. As spalling occurred at OPCP samples heated to 600 °C and 800 °C, their residual thermal strains were not measured. The spalling was not observed in OPCP during thermal strain test (as in Section 2.3.4.1) probably due to the small preload (0.1 MPa) that may restrict the uncontrolled propagation of cracks. As shown in Fig. 2.11a, slag pastes using seawater and freshwater display the same thermal strain at the temperature between 100 °C and 600 °C. However, larger shrinkage was observed in the slag paste using freshwater (FWP) exposed to 800 °C and 1000 °C. Thermal expansion observed for slag pastes from 800 °C to 1000 °C is probably caused by the sintering of geopolymer gel.





Fig. 2.11. Residual strain after elevated temperature exposure

The residual thermal strains of all the concretes are similar in the temperature range of 100 - 600 °C. In the temperature range of 800 - 1000 °C, ordinary Portland cement concrete (OPCC) has slightly greater expansion than slag concrete using freshwater and river sand (FWRSC), whilst slag concrete using seawater and sea sand has much greater expansion than OPCC and FWRSC. Thus, the use of seawater and sea sand exacerbates the residual thermal expansion when concrete is exposed to high temperature (such 800 °C and 1000 °C in this study). The sea sand is possibly the main reason for the larger residual expansion after 800 °C or 1000 °C exposure.

A comparison of the residual strains of alkali-activated slag paste, mortar and concrete utilising seawater and sea sand is shown in Fig. 2.11c. After the addition of fine aggregate, the shrinkage of the paste was restrained. The coarse aggregate leads to an increase of residual thermal strain at 800 °C and 1000 °C. The concrete shows a dramatically different thermal expansion behaviour from paste alone as the aggregates dominate the expansion. A comparison between SWSSC and SWSSC2 indicates that the coarse aggregate size does not obviously affect the residual thermal strain.

2.3.5 Mechanical properties after elevated temperature exposure

2.3.5.1 Residual strength

Residual strengths of different mixtures were obtained by the axial compression test on the samples subjected to prescribed heating and then cooling down to the room temperature. The normalized residual strength - temperature curves are shown in Fig. 2.12, in which f_c^{T} is the residual strength after exposure, f_c is the reference strength without exposure (listed in the legend box), error bars represent one time of standard deviation from the test data of three identical samples.



Fig. 2.12. Residual strength after elevated temperature exposure

The residual strength degradation trends of slag paste (SWP or FWP) is different from that of ordinary Portland cement paste (OPCP) (Fig. 2.12a). There is a rapid strength drop of SWP and FWP after exposure of 100 °C and 200 °C and the residual strength of SWP and FWP at 200 °C are only 22% and 29% of the reference strength. This is much higher (84%) for OPCP. One reason is that the cracks caused by evaporating water in slag paste is severer than that in cement paste (as suggested by its higher mass loss as shown in Fig. 2.9). The pressure caused by evaporating water leads to cracks and degrades the strength. Another reason is the slag paste sample experienced greater temperature gradient due to its lower thermal conductivity than cement paste (as discussed in Section 2.3.1). Dramatic strength deterioration of OPCP is observed from 400 °C to 600 °C because of the decomposition of portlandite (Ca(OH)₂→CaO+H₂O), which is well established in literatures (Neville 2011; Handoo et al. 2002). SWP behaves similar to FWP after elevated temperature exposure except at 100 °C. It is found that slag paste using seawater (SWP) exhibits much higher residual strength. It should be noted that FWP has a higher mass loss at 100 °C than SWP, which caused a severer strength degradation of FWP. The residual strength curves of concretes after elevated temperature exposure are summarised in Fig. 2.12b. Overall, the residual strength of concrete decreases gradually with increasing exposure temperature. There is a strength gain of OPCC from 100 °C to 200 °C, which can be attributed to the further hydration of cement (Khoury 1992). Even though the trends in residual strength change of slag paste and cement paste are different, the overall behaviours of slag concrete and cement concrete after temperature exposure are similar. It is because the strength of concrete largely depends on the interfacial transition zone and the thermal expansion incompatibility of paste and aggregates is most likely the main mechanism of the strength loss. Fig. 2.12b indicates that the slag concrete with seawater and sea sand (SWSSC) has only slightly lower residual strength than slag concrete with fresh water and river sand (FWRSC), indicating no obvious role of NaCl in influencing residual strength obviously.

A comparison of residual strength of slag-based concrete, mortar and paste using seawater is summarized in Fig. 2.12c. The mortar (SWSSM) behaves similar to paste (SWP), whilst the addition of coarse aggregate has a beneficial effect on residual strength due to the improvement in pore structure, the increase of thermal conductivity (probably improve the thermal/heat distribution in the sample) and the prevention of cracks. As mentioned previously, the mechanism of temperature induced strength reduction of paste and concrete may be different. As shown in Fig. 2.12c, the concrete (SWSSC2) that has a larger coarse aggregate size (~14 mm) has slightly lower residual strength than concrete (SWSSC) that has a smaller coarse aggregate size (~7 mm). Because the sample dimensions are different for SWSSC2 and SWSSC, the effects of aggregate size on the residual strength could not be clearly established in the current research. It is worthwhile to note that the study of Kong and Sanjayan (2010) found that the residual strength of fly ash concrete with 10-14 mm coarse aggregate is similar to that of the 20 mm coarse aggregate after 800 °C exposure.

The residual strength of geopolymer composites (paste, mortar and concrete) is affected by many factors, such as composition of raw material, alkali-activator type, alkali-activator dosage, water content (w/b), curing conditions, sample size, heating rate and exposure duration. Based on the work in this chapter and other literatures, the strength degradation after elevated temperature is the result of the combined effects of the phase changes (e.g. decomposition, geopolymerization, sintering and melting) and the cracks (that can be induced by pore pressure, temperature gradient, and thermal expansion incompatibility between paste matrix and aggregates). Though there are some reported research on this aspect, it is still difficult to quantitatively clarify the effects of those factors, and then propose strength prediction models.

Some of the existing residual strength data of alkali-activated geopolymer (slag and fly ash) composites is collected and summarized in Fig. 2.13 and Table 4. It is found that the residual strength

change trends vary significantly between different researchers' data as well as from the experimental results in this chapter (Fig. 2.12).



Fig. 2.13. Existing data of residual strength

The water evaporation occurs at relative low temperatures (<200 °C). If the moisture cannot move to sample surface freely, pore pressure may cause cracks in paste which is detrimental to residual strength. The impermeability (pore structure) and water content can affect the formation of the cracks. Based on the comparison of SEM images of fly ash paste (in Rickard et al. 2015) and slag paste (this chapter), it is found that fly ash geopolymer has more connected pores, and hence, less pressure build up during heating than slag geopolymer. The loss of capillary and chemically bonded waters (i.e. dehydration of gel) can induce shrinkage of gel and cause microcracks (Duxson et al. 2007; Mehta and Monteiro 2006; Fu et al. 2004). The temperature gradient, which is maximum at temperature lower than 300 °C (as discussed in Section 2.3.1), could also induce cracks. The gradient is affected by the conductivity of material, heating rate and sample dimensions.

The cracks are detrimental to residual strength, but some phase changes, such as geopolymerization and sintering, are beneficial to residual strength. It is found that further geopolymerization of unreacted fly ash particles can improve the residual strength of fly ash paste (Pan et al. 2009; Kong et al. 2007). At high temperatures, the sintering can change the morphology and heal cracks, which is the main reason for strength increase at temperature higher than 800 °C (as shown in Fig. 2.13, and in Rovnanik et al. 2013; Rickard et al. 2015).

The residual strength of paste is the result of the detrimental effects of cracks and beneficial effects of geopolymerization and sintering. With regard to data in Table 2.4, as the sample size is small, the influence of temperature gradient may be insignificant. Because of the effects of lower water content, better pore structure and further geopolymerization, fly ash paste experiences strength gain at temperature lower than 500 °C, whilst the strength of slag paste decreases due to its poorer pore

structure and less geopolymerization (as shown in Fig. 2.13). At ≥ 800 °C, the strength gain occurs in geopolymers due to the beneficial effects of sintering. However, the slag pastes in this chapter experienced a steep strength reduction from 23 °C to 200 °C and no strength gain was observed at 1000 °C. This could be explained by the very high water-to-binder ratio, temperature gradient, and the large cracks induced by them cannot be healed by sintering.

After the addition of aggregates, the thermal expansion mismatch (shrinkage of paste matrix and expansion of aggregates) can lead to cracks around aggregates, which has been observed in this investigation and by other researchers (Guerrieri et al. 2009; Zuda et al. 2006). The residual strength changes of concretes in this research or other studies display a similar trend (i.e. decrease gradually with increasing temperature, Fig. 2.12 and 13), which indicates that the thermal expansion incompatibility plays a dominant role in determining the residual strength even though the behaviours of pastes are different.

Label ^a Data sou		Sample		Precursor					activator			
	Data source	type	w/b ^b	Туре	SiO ₂ (wt %)	Al ₂ O ₃ (wt %)	CaO (wt %)	Si:Al (by molar)	Туре	wt % Na by precursor	Na (K):Si (by molar)	Sample size
Slag-Paste-1	Rovnanik et al. 2014	Paste	0.37	Slag	39.8	6.6	39.0	5.1	Na ₂ O·1.95SiO ₂	4.0	1.0	20×20×100 mm prism
Slag-Paste-2	Khater 2014	Paste	0.30	Slag	37.0	10.0	33.1	3.1	$Na_2SiO_3 + NaOH$	N/A	N/A	25 mm cube
Slag-Paste-3	Rashad et al. 2016	Paste	N/A	Slag	37.0	10.0	33.1	3.1	Na ₂ O*1.7SiO ₂	2.6	1.2	20 mm cube
Slag-Mortar-1	Zuda et al. 2006	Mortar	0.47	Slag	38.6	7.2	38.8	4.5	Na ₂ O·nSiO ₂	N/A	N/A	40×40×160 mm prism
Slag-Concrete-1	Guerrieri et al. 2009	Concrete	0.50	Slag	32.5	13.0	42.1	2.1	Na ₂ O·SiO ₂	4.0	2.0	D100×H200 mm cylinder
FA-Paste-1	Kong and Sanjayan 2010	Paste	0.20	Fly ash	48.8	27.0	6.2	1.5	$Na_2O \cdot 2SiO_2 + KOH$	Na: 2.6%; K: 1.9%	1.4	25 mm cube
FA-Paste-2	Pan and Sanjayan 2010	Paste	N/A	Fly ash	48.3	30.5	2.8	1.3	$Na_2O{\cdot}2SiO_2+NaOH$	N/A	N/A	D24×H48 mm cylinder
FA-Paste-3	Pan et al. 2014	Paste	0.28	Fly ash	48.3	30.5	2.8	1.3	$Na_2O{\cdot}2SiO_2+NaOH$	5.3	1.5	D24×H48 mm cylinder
FA-Paste-4	Richard et al. 2015	Paste	0.23	Fly ash ^c	50.0	24.9	1.8	1.7	$Na_2O \cdot 2SiO_2 + NaOH$	N/A	N/A	D25×H50 mm cylinder
FA-Paste-5	Richard et al. 2015	Paste	0.27	Fly ash ^d	60.0	26.6	3.3	1.9	$Na_2O{\cdot}2SiO_2+NaOH$	N/A	N/A	D25×H50 mm cylinder
FA-Concrete-1	Sarker et al. 2014	Concrete ^e	0.21	Fly ash	50.8	26.9	2.1	1.6	$Na_2O \cdot 2SiO_2 + NaOH$	4.8	1.7	D100×H200 mm cylinder
FA-Concrete-2	Sarker et al. 2014	Concrete ^f	0.21	Fly ash	50.8	26.9	2.1	1.6	$Na_2O \cdot 2SiO_2 + NaOH$	4.8	1.7	D100×H200 mm cylinder
N/A	This chapter	All ^g	0.53	Slag	34.8	13.5	41.3	2.2	Na ₂ O·1.35SiO ₂	2.8	1.5	D30×H60 mm cylinder D50×H100 mm cylinder

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Notes: a) The 'Label' of samples is in consistent with the legend in Fig. 2.13; b) w/b is water-to-binder ratio (by weight, the water in alkaline solution is included); c) fly ash was

Table 2.4. Summary of existing data on the residual strength of alkali-activated geo-polymer composites

sourced from Collie power station (denoted as 'CFA'); d) fly ash was sourced from Eraring power station (denoted as 'EFA'); e) steam curing at 60 °C for 24 h immediately after casting; f) steam curing at 80 °C for 24 h started 3 days after casting; g) it includes paste, mortar and concrete.

2.3.5.2 Typical stress-strain curves

The typical stress-strain curves of samples with and without prior heating at different temperatures are summarized in Fig. 2.14, in which the sample identification (in the format of 'temperature – number of test repeats') is presented adjacent to the corresponding curve. The axial strain was obtained from a laser-extensometer with a gauge length of ~40 mm. Due to a brittle failure of slag paste samples of '23-2' and '100-2', the descending part was not recorded. As mentioned in Section 2.3.2.2, spalling occurred during heating process for cement paste sample at 600 °C and 800 °C, and hence, their stress-strain curves cannot be obtained. A unique phenomenon of the spalling of outer layer (as shown in the photo of Fig. 2.14a) was observed during compression test on exposed slag paste samples.





Starting from 200 °C, the stress-strain curve shape of slag paste (SWP) is different from that of cement paste with the same temperature exposure. As shown in Fig. 2.14a, an obvious non-linearity is found in the initial ascending part (0 ~ $0.7f_c$ ') of the stress-strain curves of slag paste samples after exposure at \geq 200 °C. The slope of the curves within this initial part increases gradually, but the slope is constant for all the other paste or concrete samples. The explanation to the nonlinearity is the high

porosity of slag paste after water loss. Therefore, the Young's modulus of slag paste after exposure cannot be determined by the traditional method specified in AS 1012.17 (2014). With the increase of temperature, both slag paste and cement paste become deformable. The strain corresponding to the peak stress of slag paste experiences two dramatic increases: from 100 °C to 200 °C caused by water loss, and from 600 °C to 800 °C due to the occurrence of glass transition, which is also found in fly ash paste (Pan and Sanjayan 2010). On the other hand, the strain corresponding to the peak stress of cement paste does not increase as much as that of slag paste. A comparison between Fig. 2.14a and b indicates that the slag paste has a much higher ($3 \sim 10$ times higher in terms of the strain corresponding to peak stress) deformability than cement paste after temperature exposure.

The shapes of stress-strain curves of slag concrete (SWSSC2) and ordinary Portland cement concrete (OPCC) are generally similar. In the temperature range of 100 - 400 °C, the strain corresponding to the peak stress of slag concrete is slightly larger than that of cement concrete, probably due to the different behaviour of paste matrix as discussed aforementioned. However, if the temperature is above 600 °C, the strains corresponding to the peak stress of slag concrete and cement concrete are similar, even though slag paste has a much higher deformability than cement paste. The addition of aggregates makes the stress-strain behaviours of slag concrete and cement concrete similar.

2.3.5.3 Residual Young's modulus

The residual Young's modulus was obtained from the stress-strain curves (the strain was derived from displacement recorded by test machine) of samples in according with AS1012.17 (2014). As discussed in the previous section, due to the nonlinearity of the initial ascending part of the stress-strain curves of slag paste, the residual Young's modulus of paste samples shall not be discussed here. The residual Young's moduli (E_c^{T}/E_c) of concrete and mortar are plotted in Fig. 2.15, in which the values of Young's modulus of unexposed samples are also listed in the legend box (e.g. 24.4 GPa for SWSSC).

The Young's modulus decreases with increasing the exposure temperature. It is found that ordinary Portland cement concrete has a higher residual Young's modulus than slag concretes, which is in agreement with the relative relationship of residual strength between cement concrete and slag concrete (Fig. 2.12). It is known (Ding et al. 2016) that the Young's modulus of concrete can be estimated using empirical formulae suggested by codes (e.g. ACI318-11 2011) and higher compressive strength leads to higher Young's modulus. The samples after temperature exposure still obey this rule but the coefficients in the formulae may need adjustment. A comparison between Fig. 2.12 and 2.15 indicates that Young's modulus drops more rapidly than compressive strength as the

interfacial transition zone (ITZ) microcracks have a more damaging effect on the Young's modulus than compressive strength.



Fig. 2.15. Residual Young's modulus after elevated temperature exposure

2.3.6 Mechanical properties in hot condition

In this part, the compressive test was conducted on concrete samples in hot condition. Samples were heated in the furnace (Fig. 2.3) to the target temperature and held for the same duration (30 min) as for the residual strength test (Section 2.3.5). After this holding time, compressive test was carried out with specimen still in the furnace at the target temperature.

The strengths of concrete samples in hot condition (denoted as 'hot strength') are summarized in Fig. 2.16, in which the residual strength curves and a designing curve (for siliceous aggregate) suggested in BS EN 1992-1-2 (2004), are also presented. The hot strength at 100 °C is lower than the corresponding residual strength, whilst the hot strength is higher than the residual strength at > 200 °C. In this test, 200 °C seems to be a critical temperature for slag concrete at which the reduction of residual strength surpasses the reduction of hot strength. With respect of cement concrete, the hot strength at 200 °C is remarkable high, which has not been observed by other researchers, and no proper explanation has been found yet for this phenomenon. In the low temperature regime (i.e. \leq 200 °C), the water evaporation was found to continue during the compressive test. The compression forces on samples may block the pathways of vapour and increase the pore pressure, which is likely to intensify the damage. At > 200 °C, the influence of water evaporation becomes less significant and the thermal expansion incompatibility dominates the strength reduction. During cooling, the expanded aggregate tends to shrink and the cracks caused by incompatibility were severer in cooled state than in hot condition. Therefore, hot strength is higher than residual strength. Similar observations on cement concrete have been reported by researchers (Hager 2016; Bamonte and Gambarova 2009) because of the similar strength reduction mechanisms to slag concrete. It is found that slag concrete using fresh water and river sand (FWRSC) has a generally higher hot strength than

seawater and sea sand concrete probably due to the influence of sea sand. The BS EN 1992-1-2 provides an overestimation of the hot strength, especially at low temperature (i.e. ≤ 200 °C).



Fig. 2.16. Strength of concrete in hot condition

Due to the lack of available hot strength data of alkali-activated concrete, only the existing data of Portland cement concrete is summarized in Fig. 2.17, including the experimental data of Seshu and Pratusha (2013), Bamonte and Gambarova (2012), Hager (2013), Persson (2004), and the data summarized by Kodur (2014). Even though the results are rather scattered as many factors can affect the hot strength, the hot strength at 100~400 °C does not change much. The strength change trend of slag concrete in this research is similar to that of Portland cement concrete.



Fig. 2.17. Existing data of hot strength of ordinary Portland cement concrete

The Young's moduli of concrete samples in hot condition are presented in Fig. 2.18, where the residual Young's modulus is also plotted by lines. The design curves for cement concrete suggested by BS EN 1992-1-2 (2004) and BS 8110 (1985) (replotted from Buchanan and Abu 2017), and the experimental data summarized by Kodur (2014) are also presented in Fig. 18. It should be noted that as the relationship of Young's modulus to elevated temperature is not directly given in BS EN 1992-1-2 (2004), the Young's modulus data in Fig. 2.18 was derived from the stress-strain models suggested by BS EN 1992-1-2 (2004). Generally speaking, the Young's modulus in hot condition is

close to the corresponding residual Young's modulus. The Young's modulus at 100 °C is slightly lower and at 680 °C is slightly higher (except FWRSC) than the residual Young's modulus due to the same explanations for strength aforementioned. Because the Young's modulus is more sensitive to the interfacial transition zone (ITZ), the difference in Young's moduli between samples in hot and cooled conditions is less significant than that of strength. The Young's moduli of slag concretes (FWRSC and SWSSC) drop faster than Portland cement concrete and the difference between slag concrete and cement concrete is exacerbated. It is found that the experimental result in this study falls within the range (grey shaded area in Fig. 2.18) summarized by Kodur (2014). The prediction by BS EN 1992-1-2 is much conservative and the curve therefore can be regarded as the lower bound. The design curve previously suggested by BS 8110 (1985), which is commonly used in engineering practice, overestimates Young's modulus at low temperature and underestimates at high temperature.



Fig. 2.18. Young's modulus of concrete in hot condition

2.4. Macrostructures and microstructures

2.4.1 Macrostructures (X-ray CT scanning)

2.4.1.1 Air voids distribution

The macrostructures of some typcial samples before and after temperature exposure were investigated by means of X-ray CT scanning. It is known that the voids in hydrated cement paste mainly include capillary voids (10 nm \sim 5 µm), which are of irregular shape, and air voids (> 50 µm), which are generally spherical (Mehta and Monteiro 2006). Due to the limitation of the image resolution (1 pixel = 0.03 mm), the X-ray CT scanning method only can detect the air voids with dimeter larger than 30 µm.

The air void size distribution diagrams of slag paste (SWP) and concrete (SWSSC, OPCC) before exposure are shown in Fig. 2.19. The vertical axis in Fig. 2.19 (named as 'Frequency') represents the number density of air voids at a certain range of equivalent diameters determined from the middle 7.5 mm height of samples. The void ratios of SWP, SWSSC and OPCC are 0.2%, 1.1% and 1.0%

respectively. As shown in Fig. 2.19, even though the void ratio of paste is much lower than concrete, the total amount of voids in paste is much higher. Paste has much smaller voids than concrete and the voids with diameters larger than 1 mm were not observed in the paste sample. Because of the influence of aggregate, the entrapped air bubbles with large size (>1 mm) in concrete is more difficult to be expelled during compaction than those in paste. A comparison between Fig. 2.19 b and c indicates that OPCC has more small voids than SWSSC but the number density of larger voids (>1 mm) in SWSSC is higher probably due to the higher workability of OPCC. However, the overall air voids distribution trends of SWSSC and OPCC are similar.



Fig. 2.19. Air voids size distribution of unexposed samples

2.4.1.2 Cross-section image

The representative cross-section images of samples before and after 600 °C exposure are shown in Fig. 2.20. Before heating, the size of the isolated air voids in concrete samples (SWSSC and OPCC) is much larger than that in paste sample (SWP), which is in agreement with the air voids size distribution (Fig. 2.19). Minor cracks are observed near the surface of unexposed paste sample due to the drying shrinkage. It is necessary to note that the white particles in SWSSC (Fig. 2.20b) are probably the shell debris from sea sand.

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(d) SWP after exposure (e) SWSSC after exposure (f) OPCC after exposure Fig. 2.20. Cross-section images of sample before and after 600°C exposure

As shown in Fig. 2.20d, large amount of cracks are formed in slag paste sample at 600 °C, and the maximum crack width can reach about 0.3 mm. The large cracks are mainly located in the core area and propagate to the surface. During the heating stage, the pore pressure in slag paste increases as the water evaporation is partly blocked due to the lack of interconnected voids, as a results, the cracks are formed if the pore pressure exceed a certain limit (capacity of paste to resist fracture). Furthermore, the temperature gradient within the sample is another reason for the formation of cracks. The temperature in the core area of paste sample is lower than that in the outer part, especially during the period when most of the water evaporation occurs (as discussed in Section 2.3.1). The core area experienced less shrinkage than the outer layer and the cracks are formed due to the mismatch in shrinkage. As also mentioned earlier, the water content and sample size can also affect the cracks formation.

After heat exposure, obvious cracks are formed around coarse aggregates as shown in Fig. 2.20e. The cracks in paste matrix are not observed in exposed concrete samples by CT scanning. As aggregates (sand and basalt) are uniformly distributed in the paste matrix and the deformation of paste can be restrained, the influence of water evaporation and temperature gradient on cracks formation within paste matrix becomes less significant. The cracks around coarse aggregate is attributed to the different thermal expansion behaviours of paste (contraction) and basalt (expansion) as discussed in Section

2.3.4. During the cooling stage, the cracks will enlarge as most of the thermal expansion of basalt is reversible but the thermal shrinkage of paste is mostly irreversible. A comparison between Fig. 2.20 e and f indicates that the cracks in slag-based concrete using seawater and sea sand (SWSSC) are severer than those in ordinary Portland cement concrete (OPCC) as the thermal shrinkage of slag paste is larger than that of cement paste. From the point of view of cracks formation, the SWSSC would experience higher strength deterioration than OPCC after temperature exposure.

It is worthwhile to emphasize that only macro-cracks can be observed by X-ray CT scanning. The thermal-induced microcracks do exist within the paste or between the paste matrix and aggregate, which were observed by SEM (Handoo et al. 2002; Rickard et al. 2015; Fu et al. 2004).

2.4.1.3 3D configuration of air voids and cracks

The 3D configuration of air voids and cracks for unexposed and exposed samples are shown in Fig. 2.21, in which the air void size ranges are represented by different colours. The dimensions for paste samples are 30 mm dimeter and 30 mm height and those for concrete samples are 30 mm diameter and 10 mm height. The 3D images can provide a visual understanding of the air voids and cracks distribution in the samples.

The 3D images of air voids of unexposed samples provide qualitative representation of the void distribution diagrams discussed in Section 2.4.1.1. After temperature exposure, large connecting cracks are observed in paste sample that propagates from centre to surface along the entire sample (yellow colour in Fig. 2.21b). In the case of concretes, the cracks in SWSSC is severer than those in OPCC, which is in agreement with the discussion in the previous sections.


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Fig. 2.21. 3D configurations of air voids of samples before and after 600 °C exposure

2.4.2 Microstructures (SEM)

The SEM micrographs of slag paste utilising seawater (SWP) before and after exposure to different temperatures of 200, 600 and 1000 °C were presented in Fig 2.22. It should be mentioned that all the SEM samples were pre-heated at 60 °C for about 48 h to expel all moisture before SEM examination, which is essential for SEM. For unexposed slag paste, micro-cracks (right hand side of Fig. 2.22a) due to the drying shrinkage are observed. Some un-reacted slag particles appear as irregular jagged areas in the unexposed paste. A flat surface was observed in sample after 200 °C exposure (Fig. 2.22b) but the unreacted slag particles was not found. Therefore, further geopolymerization of unreacted slag particles was completed before 200 °C. Some microcracks that are larger than those seen in Fig. 2.22a were observed on the 200 °C sample, which is probably induced by temperature gradient and pore pressure. After 600 °C exposure, micropores of the order of 0.05 μ m are observed as seen in the magnified area in the inset of Fig. 2.22c. The morphology of slag paste changes greatly after exposure at 1000 °C. As shown in Fig. 2.22d, the surface becomes looser and micropores with larger scales

than those at 600 $^{\circ}$ C are observed. The vitreous phase is observed in Fig. 2.22d indicating the occurrence of sintering.

It is found that some microcracks were healed due to the reaction of unreacted slag particles and the sintering process. However, macrocracks are detected on SWP sample exposed to 600 °C using X-ray CT scanning (discussed in Section 2.4.1.2). The beneficial effects (i.e. healing of some microcracks) observed in slag paste cannot obviously improve the residual strength due to the existence of macrocracks. This can partly explain why the strength gain reported by some researchers (shown in Fig. 2.12) was not observed in this research.



Fig. 2.22. SEM images of SWP after exposure

2.5. Conclusions

In the current investigation, the thermal properties of alkali-activated slag paste, mortar and concrete utilising seawater and sea sand or freshwater and river sand were studied, and the cement-based composites were also investigated for a comparison purpose. The samples were exposed to different target temperatures including 100, 200, 400, 600, 800 and 1000 °C. The temperature gradient, visual observation, mass loss, thermal strain of samples with different mixtures were investigated. Both the residual mechanical properties after cooling and mechanical properties in hot condition were studied in this research. The X-ray CT scanning and scanning electron microscopy (SEM) were conducted to understand the macro/microstructure of samples before and after temperature exposure. The main conclusions of this research are summarized as following:

1. The temperature gradient in slag concrete (SWSSC) increases with increasing temperature and reaches the maximum at about 200~300 $^{\circ}$ C due to the influence of water evaporation.

2. The mass loss is mainly contributed by the loss of free water, physical- and chemical-bonded water. Slag pastes (SWP and FWP) exhibit a faster mass loss than cement paste, whereas the mass loss trends of concretes are similar regardless of using slag or cement. It is found that most of the mass loss occurs below 200°C and the maximum mass loss is in agreement with the water content in mixtures. The seawater and sea sand do not affect the mass loss behaviour obviously.

3. Both slag paste and cement paste experience shrinkage during heating, whilst concrete expands since the aggregates dominate the expansion during heating. Slag paste displays a higher shrinkage than cement paste, and the shrinkage of them are irreversible, which is different from the reversible expansion of concrete.

4. The residual strengths of paste, mortar and concrete decrease with increasing exposure temperature, and no strength gain was observed for slag paste in this research. The slag paste displays a rapid strength deterioration upon heating (from 100 °C), whilst the strength reduction of cement paste is less than 30% until the decomposition of portlandite (600 °C). On the other hand, the strength reduction trends of concretes are generally similar. The seawater, sea sand and coarse aggregate with larger size have a slightly (less than 10%) detrimental effect on residual strength. The samples become more deformable after heating and the residual Young's moduli drop more rapidly than residual strength when temperature is increased.

5. The hot strength of concrete is lower than corresponding residual strength at temperature below 200 $^{\circ}$ C due to the higher pore pressure led by water evaporation. However, when the temperature is higher than 200 $^{\circ}$ C, a higher strength was observed for samples in hot condition (i.e., hot strength) as the cooling exacerbates the cracks induced by the thermal expansion incompatibility.

6. With the help of X-ray CT scanning and SEM, it is found that large amount of macrocracks were formed in paste samples and around aggregates in concrete samples after temperature exposure.

Obvious morphology changes caused by geopolymerization, sintering, and crystallization were found in slag paste.

7. Based on the current study, the mechanical properties degradation of slag paste are mainly caused by cracks induced by temperature gradient and pore pressure and phase changes at high temperature, among which the cracks dominate the degradation. On the other hand, the main mechanism of the mechanical properties degradation of concrete, regardless using slag or cement, seawater or fresh water, river sand or sea sand, is the thermal expansion incompatibility between the contraction of paste matrix and expansion of aggregates. The influence of seawater and sea sand on the thermal properties is not obvious.

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References

Abdulkareem, O.A., Al Bakri, A.M., Kamarudin, H., Nizar, I.K. and ASaif A.A. (2014), Effects of elevated temperatures on the thermal behavior and mechanical performance of fly ash geopolymer paste, mortar and lightweight concrete. Construction and Building Materials, 50, 377-87.

ACI 233R-03 (2011), Slag cement in concrete and mortar, American Concrete Institute, Farmington Hills, MI.

ACI 318-11 (2011), Building code requirements for structural concrete and commentary, American Concrete Institute, Farmington Hills, MI.

Annerel, E. and Taerwe, L. (2009), Revealing the temperature history in concrete after fire exposure by microscopic analysis, Cement and Concrete Research, 39, 1239-49.

Arioz, O. (2007), Effects of elevated temperatures on properties of concrete, Fire Safety Journal, 42, 516-22.

AS 1012.17-1997 (R2014), Methods of testing concrete - Determination of the static chord modulus of elasticity and Poisson's ratio of concrete specimens, Standards Australia, Sydney.

AS 1141.11.1-2009 (2009), Methods for sampling and testing aggregates – Particle size distribution - Sieving method, Standards Australia, Sydney.

Aslani, F. (2015), Thermal performance modeling of geopolymer concrete, Journal of Materials in Civil Engineering, 28, 04015062.

ASTM C192/C192M (2007), Standard practice for making and curing concrete test specimens in the laboratory, American Society for Testing and Materials, West Conshohocken, PA.

Bamonte, P. and Gambarova, P.G. (2012), A study on the mechanical properties of self-compacting concrete at high temperature and after cooling, Materials and Structures, 45, 1375-87.

Bamonte, P. and Gambarova, P.G. (2009), Thermal and mechanical properties at high temperature of a very high-strength durable concrete, Journal of Materials in Civil Engineering, 22, 545-55.

BS 8110 (1985), Structural Use of Concrete, British Standards Institution.

BSEN 1992-1-2 (2004), Eurocode 2: Design of concrete structures – Part 1-2: General rules – Structural fire design, European Committee for Standardization.

Buchanan, A.H. and Abu, A.K. (2017), Structural design for fire safety, 2nd ed, John Wiley & Sons Ltd.

Cheng, Q.-H. and Sarkar, S.L. (1994), A study of rheological and mechanical properties of mixed alkali activated slag pastes, Advanced Cement Based Materials, 1(4), 178-184.

Collins, F.G. (1999), High early strength concrete using alkali activated slag, Monash University.

Cruz, C.R. and Gillen, M. (1980), Thermal expansion of Portland cement paste, mortar and concrete at high temperatures, Fire and Materials, 4, 66-70.

Davidovits, J. (2015), Geopolymer chemistry and applications, Geopolymer Institute.

Ding, Y., Dai, J.G. and Shi, C.J. (2016), Mechanical properties of alkali-activated concrete: A stateof-the-art review, Construction and Building Materials, 127, 68-79.

Duxson, P., Lukey, G.C. and Van Deventer J.S. (2007), Physical evolution of Na-geopolymer derived from metakaolin up to 1000 C, Journal of Materials Science, 42, 3044-54.

Fu, Y.F., Wong, Y.L., Poon, C.S., Tang, C.A. and Lin, P. (2004), Experimental study of micro/macro crack development and stress-strain relations of cement-based composite materials at elevated temperatures, Cement and Concrete Research, 34, 789-97.

Guerrieri, M., Sanjayan, J. and Collins, F. (2009), Residual compressive behavior of alkali-activated concrete exposed to elevated temperatures, Fire and Materials, 33, 51-62.

Hager, I. (2013), Behaviour of cement concrete at high temperature, Bulletin of the Polish Academy of Sciences, Technical Sciences, 61, 145-54.

Hager, I. (2014), Colour change in heated concrete, Fire Technology, 50, 945-58.

Handoo, S.K., Agarwal, S. and Agarwal, S.K. (2002), Physicochemical, mineralogical, and morphological characteristics of concrete exposed to elevated temperatures, Cement and Concrete Research, 32, 1009-18.

Katano, K., Takeda, N., Ishizeki, Y. and Iriya, K. (2013), Properties and application of concrete made with sea water and un-washed sea sand, Third International Conference on Sustainable Construction Materials and Technologies, Kyoto, Japan.

Khater, H.M. (2014), Studying the effect of thermal and acid exposure on alkali activated slag geopolymer, Advances in Cement Research, 26, 1-9.

Khoury, G.A. (1992), Compressive strength of concrete at high temperatures: A reassessment, Magazine of Concrete Research, 44, 291-309.

Khoury, G.A.(2000), Effect of fire on concrete and concrete structures, Progress in Structural Engineering and Materials, 2, 429-47.

Kodur, V. (2014), Properties of concrete at elevated temperatures, ISRN Civil Engineering, 1-15.

Kong, D.L., Sanjayan, J.G. and Sagoe-Crentsil, K. (2007), Comparative performance of geopolymers made with metakaolin and fly ash after exposure to elevated temperatures, Cement and Concrete Research, 37, 1583-9.

Kong, D.L. and Sanjayan, J.G. (2008), Damage behavior of geopolymer composites exposed to elevated temperatures, Cement and Concrete Composites, 30, 986-91.

Kong, D.L. and Sanjayan, J.G. (2010), Effect of elevated temperatures on geopolymer paste, mortar and concrete, Cement and Concrete Research, 40, 334-9.

Li, Y.L., Zhao, X.L., Raman Singh, R.K. and Al-Saadi, S. (2016a), Experimental study on seawater and sea sand concrete filled GFRP and stainless steel tubular stub columns, Thin-Walled Structures, 106, 390-406.

Li, Y.L., Zhao, X.L., Raman Singh, R.K. and Al-Saadi, S. (2016b), Tests on seawater and sea sand concrete-filled CFRP, BFRP and stainless steel tubular stub columns, Thin-Walled Structures, 108, 163-84.

Ma, Q., Guo, R., Zhao, Z., Lin, Z. and He, K. (2015), Mechanical properties of concrete at high temperature – a review, Construction and Building Materials, 93, 371-83.

Mehta, P.K. and Monteiro, P.J.M. (2006), Concrete: microstructure, properties, and materials (3rd ed.), McGraw-Hill.

Mohammed, T.U.T., Hamada, H. and Yamaji, T. (2004), Performance of seawater-mixed concrete in the tidal environment, Cement and Concrete Research, 34, 593-601.

Neville, A.M. (2011), Properties of concrete (5th ed.), Prentice Hall.

Nishida, T., Otsuki, N., Ohara, H., Garba-Say, Z.M. and Nagata, T. (2015), Some considerations for applicability of seawater as mixing water in concrete, Journal of Materials in Civil Engineering, 7, B4014004-1-7.

Pan, Z., Sanjayan, J.G. and Collins, F. (2014), Effect of transient creep on compressive strength of geopolymer concrete for elevated temperature exposure, Cement and Concrete Research, 56, 182-9.

Pan, Z., Sanjayan, J.G. and Rangan, B.V. (2009), An investigation of the mechanisms for strength gain or loss of geopolymer mortar after exposure to elevated temperature, Journal of Materials Science, 44, 1873-80.

Pan, Z. and Sanjayan, J.G. (2010), Stress-strain behaviour and abrupt loss of stiffness of geopolymer at elevated temperatures, Cement and Concrete Composites, 32, 657-64.

Persson, B. (2004), Fire resistance of self-compacting concrete, SCC, Materials and Structures, 37, 575-84.

Provis, J.L. and Bernal, S.A. (2014), Geopolymers and related alkali-activated materials, Annual Review of Materials Research, 44, 299-327.

Provis, J.L. and Van Deventer, J.S.J. (2009), Geopolymers: Structures, processing, properties and industrial applications, CRC Press, London.

Provis, J.L. (2014), Geopolymers and other alkali activated materials: why, how, and what?, Materials and Structures, 2014, 47, 11-25.

Rahier, H., Simons, W., Van Mele, B. and Biesemans, M. (1997), Low-temperature synthesized aluminosilicate glasses: Part III Influence of the composition of the silicate solution on production, structure and properties, Journal of Materials Science, 32, 2237-47.

Rama Seshu, D. and Pratusha, A. (2013), Study on compressive strength behaviour of normal concrete and self-compacting concrete subjected to elevated temperatures, Magazine of Concrete Research, 65, 415-21.

Rashad, A.M., Zeedan, S.R. and Hassan, A.A. (2016), Influence of the activator concentration of sodium silicate on the thermal properties of alkali-activated slag pastes, Construction and Building Materials, 102, 811-20.

Rickard, W.D., Kealley, C.S. and Riessen, A. (2015), Thermally induced microstructural changes in fly ash geopolymers: experimental results and proposed model, Journal of the American Ceramic Society, 98, 929-39.

Rickard, W.D., Temuujin, J. and Van Riessen, A. (2012), Thermal analysis of geopolymer pastes synthesised from five fly ashes of variable composition, Journal of Non-Crystalline Solids, 358, 1830-9.

Rovnanik, P., Bayer, P. and Rovnanikova, P. (2013), Characterization of alkali activated slag paste after exposure to high temperatures, Construction and Building materials, 47, 1479-87.

Sarker, P.K., Kelly, S. and Yao, Z. (2014), Effect of fire exposure on cracking, spalling and residual strength of fly ash geopolymer concrete, Materials and Design, 63, 584-92.

Shi, C., Roy, D. and Krivenko, P. (2006), Alkali-activated cements and concretes, CRC press, London.

Short, N.R., Purkiss, J.A. and Guise, S.E. (2001), Assessment of fire damaged concrete using colour image analysis, Construction and Building Materials, 15, 9-15.

Singh, B., Ishwarya, G., Gupta, M. and Bhattacharyya, S.K. (2015), Geopolymer concrete: A review of some recent developments, Construction and Building Materials, 85, 78-90.

Snell, C., Tempest, B. and Gentry, T. (2017), Comparison of the thermal characteristics of Portland cement and geopolymer cement concrete mixes, Journal of Architectural Engineering, 23, 04017002-1-10.

Taylor, H.F.W. (1997), Cement chemistry, Thomas Telford.

Teng, J.G., Yu, T., Dai, J.G. and Chen, G.M. (2011), FRP composites in new construction: current status and opportunities, in: Proceedings of 7th National Conference on FRP Composites in Infrastructure (Supplementary Issue of Industrial Construction), keynote presentation.

Teng, J.G. (2014), Performance enhancement of structures through the use of fibre-reinforced polymer (FRP) composites, in: Proceedings of 23rd Australasian Conference on the Mechanics of Structures and Materials (ACMSM23), Lismore, Australia, keynote presentation.

Wang, W.C., Wang, H.Y. and Lo, M.H. (2014), The engineering properties of alkali-activated slag pastes exposed to high temperatures, Construction and Building Materials, 68, 409-15.

Wang, Z., Zhao, X.-L., Xian, G., Wu, G., Singh Raman, R.K., Al-Saadi, S et al. (2017a), Long-term durability of basalt-and glass-fibre reinforced polymer (BFRP/GFRP) bars in seawater and sea sand concrete environment, Construction and Building Materials, 139, 467-89.

Wang, Z., Zhao, X.-L., Xian, G., Wu, G., Singh Raman, R.K. and Al-Saadi, S. (2017b), Durability study on interlaminar shear behaviour of basalt-, glass- and carbon-fibre reinforced polymer (B/G/CFRP) bars in sea water sea sand concrete environment, Construction and Building Materials, 156, 985-1004.

Xiao, J., Qiang, C., Nanni, A. and Zhang, K. (2017), Use of sea-sand and seawater in concrete construction: Current status and future opportunities, Construction and Building Materials, 155, 1101-11.

Zhao, R. and Sanjayan, J.G. (2011), Geopolymer and Portland cement concretes in simulated fire, Magazine of Concrete research, 63, 163-73.

Zuda, L., Pavlik, Z., Rovnanikova, P., Bayer, P. and Cerny, R. (2006), Properties of alkali activated aluminosilicate material after thermal load, International Journal of Thermophysics, 27, 1250-63.

Chapter 3

Experimental study on seawater and sea sand concrete filled GFRP and stainless steel tubular stub columns

Abstract

This chapter presents an experimental investigation on mechanical and associated properties of seawater and sea sand concrete (SWSSC) filled glass fibre reinforced polymer (GFRP) and stainless steel (SS) circular tubes. A proper SWSSC mix was developed to achieve the target strength and desirable workability. A total of 24 stub columns, including hollow sections and SWSSC fully filled tubes or double-skin tubes, were tested under axial compression with the load applied to concrete and tubes simultaneously. The stress-strain curves of the core concrete indicate that concrete strength and ductility is enhanced due to the confinement effect. Discussion focuses on the influence of tube diameter-to-thickness ratio, outer tube types and inner tube types on concrete confinement. Capacity formulae are proposed to estimate the load carrying capacity of SWSSC fully filled SS or GFRP tubes, and that of double skin tubes with four combinations of inner and outer tubes, i.e. SS and SS, SS and GFRP, GFRP and GFRP and GFRP and SS.

Keywords

Seawater sea sand concrete (SWSSC), GFRP, stainless steel, axial compression, local buckling

Nomenclature

$A_{ m c}$	Cross-section area of concrete
$A_{ m cn}$	Nominal concrete area
$A_{ m i}$	Cross-section area of inner tube
$A_{ m o}$	Cross-section area of outer tube
$A_{\rm s}$	Cross-section area of steel tube
D_{i}	Diameter of inner tube
$D_{ m o}$	Diameter of outer tube
$E_{ m h}$	Elastic modulus of GFRP in hoop direction
E_l	Elastic modulus of GFRP in longitudinal direction
$E_{ m o}$	Initial elastic modulus of stainless steel
$f_{ m av,i}$	Average stress for the inner tube
$f_{0.2}$	0.2% proof stress
$f_{\rm c}$ '	Concrete strength
fcc'	Confined concrete strength
$f_{ m ck}$	Characteristic strength of concrete
f_l	Confining stress
$f_{ m scy}$	Nominal yielding strength of composite sections
$f_{ m un}$	Nominal ultimate strength
f_{y}	Yield strength (= $_{0.2}$ for SS)
$f_{ m yi}$	Yield strength of inner tube
$f_{ m yo}$	Yield strength of outer tube
L	Specimen length

N_p	Predicted capacity
N_{t}	Test capacity
t _i	Thickness of inner tube
to	Thickness of outer tube
Δ	Axial end shortening
$\varepsilon_{\rm co}$	Ultimate strain of concrete
E _{cu}	Ultimate strain of confined concrete
$\mathcal{E}_{\mathrm{uh}}$	Ultimate strain of GFRP in hoop direction
$\mathcal{E}_{\mathrm{u}l}$	Ultimate strain of GFRP in longitudinal direction
χ	Void ratio
v	Poisson's ratio
ξ	Confinement factor
$\sigma_{ m res}$	Residual stress of GFRP tube
σ_{u}	Ultimate strength of GFRP tube

3.1 Introduction

Concrete-filled tubes (CFTs), which are composed of core concrete and encasing tubes, have been widely used in civil engineering, such as for high-rise buildings and bridge piers. CFTs exhibit large load-carrying capacity and good seismic performance mainly due to the confinement effect on core concrete provided by the encasing tube. Past researches (as summarised in Zhao et al. 2010) have indicated that the circular tubes can provide substantial strength enhancement and ductility in comparison to the square or rectangular tubes. The confinement effect of circular CFT is considered in most of the current design codes. Based on the cross-section configuration, concrete-filled tubes can be divided into fully concrete filled tubes and concrete-filled double-skin tubes.

The increase in global population (Kochhar 2014) has led to an increasing demand for resources (e.g. fresh water) and infrastructure (e.g. buildings, bridges). The huge demand of concrete, which is the most commonly used material for building infrastructure, is exacerbating the resource shortages (e.g. fresh water, river sand) and causing serious environmental impact (e.g. emission of CO2 during the production of Portland cement). One solution to these problems is to utilize seawater, sea sand, and geo-polymers (e.g. slag, fly ash) to replace fresh water, river sand and ordinary Portland cement (OPC) respectively. Another benefit of using geo-polymers is that the expansion caused by alkali silica reaction (ASR), which potentially causes concrete cracking, is considerably less in geopolymer-based concrete than in OPC-based concrete (Kupwade-Patil and Allouche 2013). The mechanical properties of alkali-activated seawater and sea sand concrete (SWSSC) are generally similar to those of conventional Portland concrete (Mohammed et al. 2004). However, conventional carbon steel tubes are not suitable to provide confinement to SWSSC because of the highly corrosive condition caused

by chloride ions of seawater in SWSSC itself (Kaushik and Islam 1995). Therefore, the stainless steel (SS) and fibre reinforced polymer (FRP) are adopted in this research due to their greater corrosion resistance.

Extensive studies have been conducted on concrete-filled carbon steel tubes (for fully filled tubes: e.g. Zhao et al. 2010, Shams and Saadeghvaziri 1997, Shanmugam and Lakshmi 2001, Han and Yang 2007, Gourley et al. 2008; for double-skin tubes: e.g. Nakanishi et al. 1999, Lin and Tsai 2005, Zhao and Han 2006, Wang et al. 2014). In recent years, there is an increasing interest in replacing carbon steel by stainless steel (SS) in marine environment due to its greater corrosion resistance. Several experimental investigations (e.g. Lam and Gardner 2008; Tam et al. 2014; Uy et al. 2011) have been conducted on fully concrete filled SS tubular columns, which indicate that the performance is quite good and current design codes are conservative for concrete-filled SS tubes. However, very little studies have been conducted on concrete-filled double-skin SS tubes (Han et al. 2011).

As a promising material, fibre reinforced polymer (FRP) is now increasingly used in concrete-filled tubes. Several studies (e.g. Teng and Lam 2004; Ozbakkaloglu et al. 2013) have been carried out on concrete-filled FRP wraps (with fibres exclusively oriented in hoop stress direction) and some stress-strain models have been proposed for the FRP wrap confined concrete (Teng and Lam 2004; Teng et al. 2013). In recent years, some researchers (e.g. Fam and Rizkalla 2001; Zhang et al. 2015) also looked into fully concrete filled FRP tubes (with fibres oriented both in hoop and longitudinal directions) for the use of tubes as formwork. To the best of authors' knowledge, only one experimental study (Fam and Rizkalla 2001) has been conducted on concrete-filled double-skin tubes (using specimens with FRP as both outer and inner tubes) but no study on concrete-filled double-skin tubes (FRP as outer and SS as inner tube) is reported.

This chapter reports an overall experimental investigation on seawater and sea sand concrete (SWSSC) filled circular tubular columns, including SWSSC fully filled tubes and double-skin tubes with different combinations of tube materials (stainless steel (SS) or glass fibre reinforced polymer (GFRP)). Firstly, a proper SWSSC mix was developed to achieve the target strength and desirable workability. The material properties of stainless steel and GFRP were determined by standard tensile tests. Axial compressive test was conducted on a total of 24 stub columns, including SWSSC-filled SS tubes, SWSS-filled GFRP tubes and corresponding hollow section tubes. An understanding of comparative properties has been developed based on the existing theories and the test results of this study. Finally, new methods are proposed to estimate the strength of SWSSC-filled SS tubes and GFRP tubes. It is worthwhile to mention that this chapter forms part of a large research program on hybrid SWSSC construction being carried out at Monash University in collaboration with The Hong

Kong Polytechnic University and Southeast University, China. In the next stage, the SWSSC-filled tubes will be immersed in seawater for different durations to assess the influence of corrosive environment.

3.2 Experimental investigation

3.2.1 Specimens

A total of 24 circular stub columns, including 8 hollow tubes, 8 SWSSC fully filled tubes, and 8 SWSSC-filled double-skin tubes, were prepared and tested in the present study (Fig. 3.1). The specimens were made of seawater sea sand concrete (SWSSC), or stainless steel (SS) tube, and /or GFRP tubes. Four sizes of tubes (with nominal diameter of 50 mm, 101 mm, 114 mm, and 165 mm and with nominal thickness of 3 mm) were used for the specimens and the length of all the specimens was around 400 mm long which avoided the global buckling and the influence of end effect.

Specimen	Outer t	ube (m	m)	Inner tu	Nt		
specifien	$D_{\rm o}$	to	Mat.	D_{i}	ti	Mat.	(kN)
S50-H	47.9	2.79	SS	N/A	N/A	N/A	118
S101-H	101.2	2.81	SS				335
S114-H	114.0	2.86	SS				355
S165-H	168.3	3.23	SS				545
F50-H	51.2	3.20	GFRP				98
F101-H	100.2	2.94	GFRP				199
F114-H	115.3	3.03	GFRP				206
F165-H	158.0	2.96	GFRP				213
S50-C	47.9	2.77	SS				199
S101-C	101.2	2.83	SS				729
S114-C	113.9	2.88	SS				800
S165-C	168.2	3.15	SS				1522
F50-C	51.1	3.07	GFRP				244
F101-C	100.1	3.13	GFRP				670
F114-C	115.2	3.13	GFRP				813
F165-C	158.2	3.14	GFRP				1336
S114-S50-C	114.5	2.87	SS	47.9	2.73	SS	909
S165-S101-C	167.8	3.18	SS	101.2	2.80	SS	1409
S114-F50-C	114.2	2.95	SS	51.2	3.20	GFRP	799
S165-F101-C	168.4	3.22	SS	100.3	3.06	GFRP	1167
F114-S50-C	114.8	2.91	GFRP	47.9	2.82	SS	795
F165-S101-C	158.0	2.92	GFRP	101.8	2.91	SS	880
F114-F50-C	114.7	2.93	GFRP	51.3	3.09	GFRP	872
F165-F50-C	158.3	3.13	GFRP	100.3	3.13	GFRP	1301

Table 3.1. Details of specimens



(a) Tubes fully filled with concrete Fig. 3.1. Type of cross-sections

The dimensions of the test specimens are presented in Table 3.1, where the failure loads (N_t) are also given. The label of specimen consists of outer tube material ("S" for stainless steel and "F" for GFRP), outer tube nominal diameter ("50", "101", "114", and "165"), inner tube material (only for double-skin tubes), inner tube nominal diameter (only for double-skin tubes), and cross-section type indicator ("H" for hollow section and "C" for concrete-filled section). For example, S114-C refers to fully SWSSC-filled stainless steel tube with D_0 of 114 mm, and S114-F50-C refers to SWSSC-filled double-skin tube with an outer stainless steel tube (D_0 of 114 mm) and an inner GFRP tube (D_i of 50 mm).

3.2.2 Material properties

3.2.2.1 Seawater and sea sand concrete (SWSSC)

Alkali activated slag concrete with seawater and sea sand was used in this research. The 3% (percentage weight of slag) sodium meta-silicate activator, which is composed of 47% SiO₂ and 36% Na₂O, was pre-blended with slag in the dry form before mixing. The seawater and sea sand were obtained from Brighton beach in Melbourne. The chemical composition of the seawater is: Na (11940 mg/L), Mg (1430 mg/L), K (622 mg/L), Cl (20700 mg/L), SO₄ (3420 mg/L), and that of the sea sand is (weight percentage): SiO₂ (96.5%), CaO (1.3%), Cl (0.13%), SO₄ (0.01%). Sieve analysis was carried out according to AS1141.11.1 (2009) to determine the particle size distribution (PSD) of sea sand. The PSD for the sea sand in this study is compared in Fig. 3.2 with those of river sand and desert sand reported in Chuah et al. (2016). The fineness of sea sand is between those of river sand and desert sand. Another parameter to describe the fineness of sand is so-called fineness modulus which is an empirical factor obtained by adding the cumulative percentages of aggregate retained on each of the standard sieves ranging from 80 mm to 150 micron and dividing this sum by 100. The larger the fineness modulus, the coarser is the sand. The measured fineness modulus of the sea sand in the current study is 2.39, which is similar to that of some sea sand in China reported in Huang

(2007) with a fineness modulus ranging from 1.98 to 2.85 and in Nong (2008) with a fineness modulus of 2.43.

The coarse aggregate consisted of 14 maximum size basalts with a specific gravity of 2.95. In order to improve the concrete workability, 1% (percentage weight of slag) hydrated lime slurry was added with water. As a result, the slump of fresh concrete could reach 160 mm. The concrete mixture proportions are reported in Table 3.2 and the seawater-to-slag ratio is 0.53.



Table 3.2. Concrete mixture

Constituents	weight (kg/m ³)
Slag	360
Seawater	190
Sea sand	830
Coarse aggregate	1130
Sodium meta-silicate	38.4
Hydrated lime slurry	14.4

Six identical concrete cylinders with diameter of 100 mm and height of 200 mm were cast for measuring compressive strength. All the specimens and cylinders were cured in the curing chamber with relative humidity > 90% and temperature of 20°C before testing for 28 days. The averaged 28-day strength (f_c) was 31.4 MPa.

3.2.2.2 Stainless steel (SS)

The SS tubes are 316 grade austenitic stainless steel in accordance with AS/NZS 4673 (2001). The ends of the tensile coupons were flattened in order to be gripped by the test machine. The tensile coupon test was conducted in accordance with AS 1391 (2007) with a loading rate of 1.0 mm/min. The averaged test data are summarised in Table 3.3, where $f_{0.2}$ is 0.2% proof stress, f_u is the ultimate strength. The averaged initial elastic modulus (E_0), total elongation and Ramberg-Osgood parameter (n) are 178 GPa, 55%, and 8.3 respectively. It should be noted that the $f_{0.2}$, E_0 , and n are determined

based on strain gauges, which are attached on both the concave and convex sides of tensile coupons. The typical full-range stress-strain curve (from extensometer) in which the abscissa is named as "Strain (full range)" and the stress-strain curve during the initial stage (from averaged strain gauge readings) in which the abscissa is named as "Strain (initial range)" are plotted in Fig. 3.3.

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Tube	$f_{0.2}$ (MPa)	f _u (MPa)						
50×3	306.8	618.2						
101×3	324.4	647.2						
114×3	270.3	579.4						
165×3	280.1	575.3						

Table 3.3. Tensile coupon test results of stainless steel



Fig. 3.3. Typical stress-strain curves of tensile stainless steel coupon

3.2.2.3 Glass fibre reinforced polymer (GFRP)

The GFRP tubes were fabricated by filament winding process with different glass fibre orientations. Based on the manufacturer data, 20%, 40%, and 40% fibres were in the angles of 15° , $\pm 40^{\circ}$ and $\pm 75^{\circ}$ with respect to longitudinal axis of tubes. Therefore, the GFRP tube can provide strength and stiffness in both hoop and longitudinal directions.

Tensile coupon test was conducted to evaluate the GFRP properties in longitudinal direction. As the ends of coupon were not flat, two sets of gripping pieces made of aluminium were used to ensure the gripping head of test machine did grip the coupon specimen tightly (Fig. 3.4 (a)). A total of 4 strain gauges were attached at the middle part of each coupon: on the convex and concave sides, as well as in the longitudinal and transverse directions. The averaged stress-strain curve is shown in Fig. 3.5, in which the strain is the averaged value of longitudinal strain gauge readings on both sides. As shown in Fig. 3.5, the stress-strain curves display certain nonlinearity after the strain exceeds 0.003 which is mainly caused by the failure of resin matrix. The results of tensile coupon tests are summarised in

Table 3.4, in which f_{ul} is the ultimate strength, E_l is the elastic modulus calculated based on ASTM-D3039 (2014), v is Poisson's ratio, and ε_{ul} is the ultimate strain.



Table 3.4. Material properties of GFRP

Tube size	Longitudi	Hoop direction			
Tube size	f_{ul} (MPa) ε_{ul} E_l (GPa		E_l (GPa)	ν	fuh (MPa)
50×3	242.4	0.020	23.2	0.32	272.9
101×3	229.3	0.026	18.7	0.30	368.0
114×3	211.7	0.018	19.2	0.33	328.3
165×3	186.8	0.012	20.7	0.32	266.0
Mean	217.6	0.019	20.1	0.32	308.8
COV	0.11	0.31	0.10	0.08	0.16

The hoop strength of GFRP tubes was obtained using the "disk-split" method, which is similar to the test method used in ASTM D2290 (2012). Two 13 mm wide rings were cut from each size of GFRP tubes. The test setup is shown in Fig. 3.4 (b), which consisted of U-shaped headers and semi-circular steel cylinders with the same curvature as the tested rings. All the rings failed at the gap area between the semi-circular cylinders.

In order to obtain the elastic modulus, 3 strain gauges were installed on the test rings, among which one was centered at gaps, and the others were located at 25 mm and 50 mm away from the gap. Due to the bending effect around the gap, the strain gauge reading around the gap may not be reliable. Hence the strain is taken as the averaged reading of the other two strain gauges away from the gap for generating stress-strain curves in Fig. 3.5 (in which the stress was obtained by dividing the applied force by two times the cross-section area of the GFRP ring. Fig. 3.5 shows the stress-strain curve in hoop direction to have less nonlinearity than that in longitudinal direction. The ultimate strength of GFRP in hoop direction (f_{uh}) is reported in Table 3.4. The average elastic modulus (E_h) in hoop direction is 25.2 GPa, which is higher than that in longitudinal direction (20.1 GPa) and the average ultimate strain (ε_{uh}) in hoop direction is 0.014, which is lower than that in longitudinal direction (0.019).



Fig. 3.5. Average stress-strain curves of tensile GFRP coupons

3.2.3 Mechanical test setup

The concrete-filled specimens were tested on a 5000 kN capacity Amsler machine, whilst all the hollow section specimens were tested on a 500 kN capacity Baldwin machine. The axial compressive load was directly applied on the specimens. For concrete-filled specimens, cement paste was used to fill the gap caused by concrete shrinkage to assure the load was simultaneously applied on both tube and concrete. The loading rate is 0.5 mm/min with displacement control.

Three linear variable displacement transducers (LDVTs) were equally placed around tested tubes and their averaged data were used to estimate the axial end shortening. Three longitudinal and three circumferential strain gauges were affixed to all columns at mid-height. All the loads, displacements and strains were automatically recorded by a data acquisition system. A typical test setup illustration is shown in Fig. 3.6.



(b) Front view (c) Photo Fig. 3.6. Test setup and instrumentation (e.g. S114-C)

3.2.4 Test results

3.2.4.1 Hollow sections

The failure modes of stainless steel hollow section under axial compression were slightly different due to the differences in the length-to-diameter ratios (L/D_o) and diameter-to-thickness ratios (D_o/t). Specimen S50-H, being the most slender specimen, failed by both global and local buckling, while the other specimens failed only by local buckling (e.g. S114-H shown in Fig. 3.7).



(a) S50-H (b) S114-H Fig. 3.7. Failure modes of stainless steel hollow sections



Fig. 3.8. Stress-strain curves comparison of hollow sections and tensile coupons

A comparison of stress-strain curves of hollow sections and tensile coupons is shown in Fig. 3.8, in which the strain for hollow sections is based on the ratio of axial end shortening to specimen length, whereas the strain for tensile coupons is the averaged strain gauge readings. Because of the occurrence of global buckling, specimen S50-H could not fully utilize its strength and failed much earlier than the other specimens. As shown in Fig. 3.8, with the increase of diameter-to-thickness ratio (i.e. from S101-H to S165-H), the strain corresponding to peak load decreases as the section becomes more

slender. During the initial stage (strain < 0.5%), the stress-strain curves of hollow sections (except S50-H) and tensile coupons are fairly close to each other. Then, the increase in load of hollow section became lower than that of tensile coupons as the elastic-plastic local buckling happened. It is necessary to mention that the stress-strain relationships of SS under tension and compression are different but this difference is insignificant and was ignored in this study.

Fig. 3.9 shows the stress-strain curves for GFRP tubes under axial compression. The load increased linearly until the local buckling (mainly caused by fibre's buckling) along the longitudinal direction near the ends. After this sudden failure, the GFRP hollow section still had a residual strength and could sustain the load until the axial strain reached about 8%. Based on the stress-strain curves of tested specimens, a simplified stress-strain model of GFRP hollow section is proposed in Fig. 3.10, in which σ_u is the ultimate compressive strength, ε_u is the ultimate strain and σ_{res} is the residual strength. The ultimate compressive strength, ultimate strain and residual strength of the hollow sections are summarised in Table 3.5 and the average tensile strength (*f*_{ul}) of GFRP coupons are also given in Table 3.5. As shown in Table 3.5, the ultimate compressive strength is slightly lower than the tensile strength of GFRP due to the influence of local buckling.

Table 3.5. Test results of GFRP hollow sections

Specimen	$\sigma_{\rm u}$ (MPa)	Eu	$\sigma_{\rm res}$ (MPa)	f_{ul} average (MPa)
F50-H	202.9	0.012	111.6	
F101-H	221.4	0.012	53.9	217ϵ
F114-H	192.5	0.011	43.7	217.0
F165-H	148.1	0.009	32.6	



Fig. 3.9. Stress-strain curves of GFRP hollow sections



Fig. 3.10. Simplified stress-strain model for GFRP hollow sections

3.2.4.2 SWSSC-filled tubes

Based on the material type of tubes, the SWSSC-filled tubes are divided into two groups: concrete-filled stainless steel tubes and concrete-filled GFRP tubes. Each group includes both fully filled tubes and double-skin tubes. All the load-axial strain curves are summarised in Fig. 3.11. In general, the bilinear response of SWSSC-filled GFRP tubes is much more obvious than that of SWSSC-filled SS tubes, while SWSSC-filled SS tubes display much higher ductility.



(a) SWSSC-filled stainless steel tubes (fully filled and double-skin with SS as the outer tube)



(b) SWSSC-filled GFRP tubes (fully filled and double-skin with GFRP as the outer tube) Fig. 3.11. Load-strain curves of concrete-filled tubes

The failure modes for concrete-filled stainless steel tubes (except S50-C) was an outward folding failure mechanism (local buckling) as shown in Fig. 3.12(a-b), which are in agreement with other researchers' observation (e.g. Lam and Gardner 2008; Uy et al. 2011). Due to the large length-todiameter ratio (L/D₀), specimen S50-C failed by global and local buckling. For double-skin tubes (SS as outer and GFRP as inner tube), there are sudden drops of applied load due to the buckling of inner GFRP tube (Fig. 3.12(c). The load-strain curves for SWSSC-filled SS tubes indicate that after reaching peak load, the tubes can still sustain the load which displays high ductility (Fig. 3.12(a-c)). The test was terminated due to the limited stroke of test machine (about 60 mm). It is necessary to mention that there is obvious load drop after peak load for double-skin tubes with large void ratio (e.g. S165-S101-C) as a result of buckling of the inner tube. As shown in Fig. 3.12(a-c), similar behaviour is found in stress-strain relationship for fully filled tubes and double-skin tubes (except the bulking of the inner GFRP tube).





(c) Double-skin tubes (SS as the outer tube and GFRP as the inner tube)



Fig. 3.12. Load-strain curves and failure modes

The failure of SWSSC-filled GFRP tubes was caused by the GFRP tube rupture in hoop direction (Fig. 3.12(d-e)). Before reaching the ultimate load, the GFRP tube first buckled in longitudinal direction, which led to a sudden drop of applied load. Based on the test observation, the buckling of the inner GFRP tube in SWSSC-filled double-skin tubes took place earlier than that of outer GFRP tube. It should be mentioned that the GFRP tube buckling in longitudinal direction happened more than once during the loading process (as indicated by the load-strain curves in Fig. 3.12(d-e)). There is not much difference in the shape of stress-strain curves for fully filled tubes and double-skin tubes. Compared to SWSSC-filled stainless steel tubes, the ultimate strain of SWSSC-filled GFRP tubes is much lower and they display a "brittle" behaviour.

As shown in Fig. 3.10(a), the load-strain curves of SWSSC-filled stainless steel tubes in this research did not exhibits much strain-hardening responses. These load-strain curves can be classified as "Type B" curve based on Uy's research (2011) and the tested failure load (N_t) is taken as the first peak load during the test. The tested failure load of SWSSC-filled GFRP tube is the maximum load during the test, which is also the load at which GFRP tube rupture happens. The tested failure loads of all specimens are summarised in Table 3.1.

3.3 Discussion

3.3.1 Axial strain

The axial strain can be obtained in two ways: (1) the averaged reading from axial strain gauges at mid-height (called "localised strain", ε) and (2) the ratio of axial end shortening to specimen length (called "overall strain", Δ/L). The localised strain can represent the strain of outer tubes and the overall strain can be regarded as the average strain throughout the whole length of the specimen. The latter is closer to the strain of confined concrete.

Comparisons of the localised strain and the overall strain of some specimen configurations of SS and GFRP are summarised in Fig. 3.13, which also consists the corresponding non-dimensional load (a ratio of applied load to the maximum load) versus strain (N/N_t - ε) curves. Fig. 3.13 indicates that: (1) at the initial loading stage, because of the possible gaps between loading plate and specimen end, the value of Δ/L can be much higher than ε ; (2) when stainless steel tube is experiencing large deformation, such as elephant foot, Δ/L is much higher than ε ; (3) For specimens containing GFRP tubes, once GFRP tube buckles, the Δ/L is much different from ε . As shown in Fig. 3.13, the value of (Δ/L)/ ε is more or less around 1.0 after the initial loading stage and before the collapse of the specimen.



(b) Fully filled SS tube and double-skin tube (SS as both inner and outer tubes)



(c) Double-skin tube (SS as the outer tube and GFRP as the inner tube)



(d) Fully filled GFRP tube and double-skin tube (GFRP as the outer tube) Fig. 3.13. Comparison between overall strain (Δ/L) and localized strain (ε)

The overall strain (Δ/L) is adopted as the axial strain of specimens throughout this chapter. Nevertheless, it should be emphasised that the axial strain probably cannot represent the real strain of confined concrete after experiencing large deformation.

3.3.2 Ultimate strain of GFRP tube

The ultimate longitudinal strain is taken as the overall strain at which first buckling appeared in GFRP tube in longitudinal direction and the results are summarised in Table 3.6. Except specimen S114-F50-C, the ultimate longitudinal strain of GFRP inner tube is in good agreement with that of corresponding hollow sections. For fully filled tubes, the ultimate longitudinal strain of GFRP tube is slightly higher than that of corresponding hollow sections. The ultimate longitudinal strain of GFRP outer tube in double-skin tubes is much higher than that of corresponding hollow sections. It is believed that the filled-in concrete can delay the occurrence of local buckling of GFRP outer tubes. Furthermore, the stress condition in the tubes probably also affects the local buckling. The GFRP inner tube is under longitudinal compression and hoop compression, while the outer tube is under longitudinal compression.

Specimen	Inner	Corresponding	Ratio of inner to	Outer	Corresponding	Ratio of outer to
·· I · · · ·	Tube	hollow section	hollow	tube	hollow section	hollow
F50-C	N/A	N/A	N/A	0.015	0.012	1.26
F101-C	N/A	N/A	N/A	0.012	0.012	1.00
F114-C	N/A	N/A	N/A	0.014	0.011	1.26
F165-C	N/A	N/A	N/A	0.011	0.009	1.26
F114-F50-C	0.012	0.012	1.00	0.018	0.011	1.62
F165-F101-C	0.013	0.012	1.08	0.024	0.009	2.79
S114-F50-C	0.019	0.012	1.62	N/A	N/A	N/A
S165-F101-C	0.010	0.012	0.78	N/A	N/A	N/A
F114-S50-C	N/A	N/A	N/A	0.016	0.011	1.38
F165-S101-C	N/A	N/A	N/A	0.021	0.009	2.47

Table 3.6. Ultimate longitudinal strain of GFRP tube

The ultimate hoop rupture strain of GFRP tube is taken as the averaged reading of strain gauges in hoop direction and the results are summarised in Table 3.7, in which the rupture strain obtained from material test (disk-split test) is also listed. The ultimate hoop rupture strain of GFRP tube agrees well with that from disk-split test. This conclusion is different from that for concrete-filled FRP wraps, in which FRP hoop rupture strain in confined cylinders is much lower than that from flat coupon test (Lam and Teng 2004). The reason for the difference is that the hoop strength of GFRP tube in the present study is obtained by disk-split test on GFRP rings whereas the hoop strength of FRP wrap (Lam and Teng 2004) was from flat coupon test.

Specimen	Outer tube
F101-C	0.013
F165-C	0.013
F114-F50-C	0.012
F165-F101-C	0.008
F114-S50-C	0.012
F165-S101-C	0.014
Average hoop strain from disk-split test	0.014

Table 3.7. Ultimate hoop rupture strain of GFRP tube

3.3.3 Axial stress-strain curves of concrete

3.3.3.1 Load distribution

It is well known that due to the bi-axial stress condition, the confined concrete strength (f_{cc} ') is higher than that of unconfined concrete strength (f_{c} '). In order to discuss the confinement effect, the stressstrain curves of the concrete should be determined. As the tubes and concrete resist the applied load simultaneously, the load distribution among them will be first discussed here.

The load carried by the concrete is assumed equal to the difference between the applied load and the load carried by the tubes at the same strain. The load carried by stainless steel tube can be determined based on the stress-strain curve of corresponding hollow sections. If the strain of the concrete-filled tubes exceeds ultimate strain of corresponding hollow section, the load carried by stainless steel tube is assumed to be equal to the ultimate load of corresponding hollow section (as shown in Fig. 3.14). The load carried by the outer GFRP tube is calculated by multiplying the secant elastic modulus (obtained from GFRP hollow section test, 17.3GPa) with axial strains of GFRP tube (from strain gauges). The load carried by the inner GFRP tubes is determined by the simplified stress-strain model (Fig. 3.10) of corresponding hollow sections. The load distribution curves of typical test specimens are plotted in Fig. 3.15.



Fig. 3.14. Stress-strain model for stainless steel tube



The specimen S50-H failed by global and local buckling, but the global buckling of inner SS tube with diameter of 50 mm in specimen S114-S50-C and F114-S50-C was avoided due to the filled-in concrete. Therefore, the stress-strain curves of S50-H cannot be used to calculate the load distribution in S114-S50-C and F114-S50-C. For specimen F50-C and F114-C, the strain gauges went out of function before specimen reaching peak load. The stress-strain curves of specimen S50-C, S114-S50-C, F114-S50-C, F50-C and F114-C are excluded in the discussion.

The stress in the concrete is equal to the load resisted by concrete divided by concrete area. It is emphasized that because the load in tube is derived from axial strain and the buckling of GFRP tube led to a sudden drop of applied load, the stress-strain curves in concrete are probably not accurate enough. Nevertheless, the relative comparison between them is still helpful in understanding the influence of some key parameters on the behaviour of confined-concrete.

3.3.3.2 Effects of tube diameter to thickness ratio

As shown in Fig. 3.16, with the increase of tube diameter to thickness ratio, the concrete stress to unconfined concrete strength ratio (σ_c/f_c ') decreases. This is consistent with previous findings (Zhao et al. 2010), i.e. the increase of tube diameter to thickness ratio can lead to the decrease of confining pressure acting on the concrete, which decreases the confined concrete strength. Fig. 3.16 also indicates that the concrete enhancement caused by SS tube is slightly lower than that by GFRP tube, but the confinement lasts for larger axial strain.





As shown in Fig. 3.17, the types of outer tube (SS tube or GFRP tube) can obviously affect the shape of σ_c/f_c '-strain curves. The strength enhancement caused by GFRP tube is more significant than that by stainless steel tube, but the ultimate strain in SWSSC-filled GFRP tube is much lower than that in SWSSC-filled stainless steel tube. The σ_c/f_c '-strain behaviour of SWSSC-filled GFRP tube is

generally characterised by a bilinear response. When the expansion of concrete exceeds that of GFRP tube, the GFRP tube is fully activated in confinement and the confining pressure increases continually until the hoop rupture of GFRP tube. This kind of confinement effect is called as "active confinement effect". The behaviour of SWSSC-filled stainless steel tube is slightly different. After the stainless steel tube reaches yielding strength (0.2% proof strength), the increase of confining pressure slows down (for carbon steel with yielding plateau, the confining pressure will keep constant). This kind of confinement effect is called as "passive confinement effect". The different confinement behaviour of stainless steel tube and GFRP tube is substantially attributed by their different material properties.



Fig. 3.17. Effects of the outer tube types on σ_c/f_c '-strain curves

3.3.3.4 Effects of inner tube types

The inner tube in SWSSC-filled double-skin tubes can effectively restrain the inward expansion of the concrete. Research by Fam & Rizkalla (2001) indicated that the addition of inner tube could enhance the confinement effect.



Fig. 3.18. Effects of the inner tube type on σ_c/f_c '-strain curves

As mentioned before, the buckling of inner GFRP tube can cause a sudden drop of applied load and the type of inner tube can affect the shape of load-strain curves (as shown in Fig. 3.18). Fig. 3.18

shows that when the outer tube is GFRP the influence of inner tube type is negligible. However, when the outer tube is stainless steel, the ultimate stress is similar, but the inner SS tube will result in a much more ductile behaviour than that with GFRP inner tube.

3.4 Capacity prediction of SWSSC-filled tubes

3.4.1 Capacity prediction of SWSSC fully filled stainless steel tubes

Extensive research has been conducted on concrete-filled carbon steel tubes (Zhao et al. 2010). A design method was documented in detail by Han et al (2005) where a confinement factor (ξ) was adopted to address the passive confinement of carbon steel tube on concrete. The term ξ can be determined by Eq. (3.1):

$$\xi = \frac{A_s f_y}{A_c f_{ck}} \tag{3.1}$$

where A_s is the cross-section area of steel tube, f_y is the yield strength of steel, A_c is the cross-section area of concrete, f_{ck} is the characteristic strength of concrete. As given in Han et al. (2005), the predicted load carrying capacity (N_p) of concrete-filled circular stub columns can be determined by Eq. (3.2) and Eq. (3.3):

$$N_p = (A_s + A_c) f_{scy} \tag{3.2}$$

$$f_{scy} = (1.14 + 1.02\xi) f_{ck} \tag{3.3}$$

where f_{scy} is the "nominal yielding strength" of composite sections. The relationship between f_{scy} and f_{ck} (i.e. Eq. (3.3)) is obtained by using the regression analysis method.

Comparison to carbon steel, the major difference of stainless steel is its rounded-shape stress-strain curves without obvious yielding plateau. Therefore Eq. (3.3) cannot be directly used to determine the capacity of concrete-filled stainless steel tube. Furthermore, in some countries (e.g. US, Australia), the concrete cylinder strength (f_c ') is more widely used in design than concrete characteristic strength (f_{ck}).

Eq. (3.1) can be rewritten as:

$$\xi = \frac{A_s f_y}{A_c f_c} \tag{3.4}$$

and the new relationship between f_{scy}/f_c ' and ξ is obtained by regression analysis method. The test data used in regression analysis (Lam and Gardner 2008; Tam et al. 2014; Uy et al. 2011; Yang and Ma 2013) are summarised in Table 3.8. The regression analysis result is summarised in Fig. 3.19.

Therefore the compression capacity of circular concrete-filled stainless steel stub column can be determined by Eq. (3.5) and Eq. (3.6):

$$N_p = (A_s + A_c) f_{scy} \tag{3.5}$$

$$f_{scy} = (1.14 + 1.4\xi)f_{c}^{'} \tag{3.6}$$

The comparison between the test capacity N_t and predicted capacity by this modified method is shown in Fig. 3.20 for all the data collected. As shown in Fig. 3.20, the modified method can estimate the capacity accurately with an error generally less than 15%. The comparison for SWSSC filled SS tubes is shown in Table 3.9, which suggests a good agreement.



Fig. 3.19. Regression analysis of existed data



Fig. 3.20. Comparison between predicted capacity and test capacity

Table 3.8. Test data of concrete-filled stainless steel tube for regression analysis

$D_{ m o}/t_{ m o}$	<i>f</i> _{0.2} (MPa)	fc' (MPa)	No. of specimens	Reference	Notes
64-102	259-320	20, 30	7	Uy et al. 2011	Only "type B" specimens
19, 52	266, 412	31-65	6	Lam & Gardner 2008	"Ntest(5%)" is used as tested capacity
59	340	38-42	4	Tam et al. 2014	Recycled aggregate were used
68	287-293	44-50	7	Yang & Ma 2013	Recycled aggregate were used
36-53	270-324	31.4	3	This chapter	

Data	Specimen	$N_{\rm t}({\rm kN})$	$N_{\rm p}({\rm kN})$	N_p/N_t
	S101-C	729	734	1.01
	S114-C	800	786	0.98
This	S165-C	1522	1487	0.98
enapter	Mean			0.99
	COV			0.01
All the data	Mean	1.00		
listed in Table 3.8	COV			0.13

Table 3.9. Comparison between experimental capacity and estimated capacity for SWSSC filled stainless steel tubes

3.4.2 Capacity prediction of SWSSC-filled double-skin tubes (SS as the outer tube)

The studies on concrete-filled double-skin stainless steel tube are rather limited. Han et al. (2011) investigated the behaviour of double skin tubes with SS as the outer tube and carbon steel as the inner tube. They found that the outer tube behaves like a tube fully filled with concrete, whereas the inner tube behaves like an empty one without local buckling when the void ratio (χ) is less than 0.8. This is similar to that observed for double skin tubes with carbon steel as both outer and inner tubes (Tao et al. 2004). A design capacity model was presented in Han et al. (2011) for double skin tubes, which is a summation of the capacity of outer steel tube with sandwiched concrete and the capacity of inner tube. The formulae in Han et al. (2011) are adopted here with some modifications, i.e. replacing *f*_{ck} by *f*_c' and replacing the term (1.14+1.02 ξ) by (1.14+1.4 ξ) as derived in Section 3.4.1.2 of this chapter.

The modified formulae are listed below:

$$N_{p} = (A_{o} + A_{c})f_{scy} + A_{i}f_{yi}$$
(3.7)

$$f_{scy} = \frac{\alpha}{1+\alpha} \chi^2 f_{yo} + \frac{1+\alpha_n}{1+\alpha} (1.14+1.4\xi) f_c'$$
(3.8)

$$\xi = \frac{A_o f_{yo}}{A_c f_c} \tag{3.9}$$

$$\alpha = A_o / A_c \tag{3.10}$$

$$\alpha_n = A_o / A_{cn} \tag{3.11}$$

$$\chi = D_i / D_o \tag{3.12}$$

where A_0 is the outer tube cross-section area, A_c is the concrete area, A_i is the inner tube cross-section area, A_{cn} is the nominal concrete area (= $\pi D_0^2/4$), f_{yo} is the yield strength of the outer tube ($f_{0.2}$ for stainless steel), f_{yi} is the yield strength of inner tube ($f_{0.2}$ for stainless steel or residual strength, σ_{res} for GFRP defined in Fig. 3.10), f_c ' is the concrete strength and χ is the void ratio.

A comparison between the test results and prediction using Eqs. (3.7-12) is summarised in Table 3.10. As shown in Table 3.10, the modified formula can be extended to estimate the capacity of double skin tubes with SS as the outer tube and SS or GFRP as the inner tube.

Specimen	fyo (MPa)	f _{yi} (MPa)	fc' (MPa)	χ	ξ	$N_{\rm t}$ (kN)	$N_{\rm p}({\rm kN})$	N_p/N_t
S114-S50-C	270.3	306.8	31.4	0.42	0.93	909	803	0.88
S165-S101-C	280.1	324.4	31.4	0.60	0.72	1409	1352	0.96
S114-F50-C	270.3	113.6	31.4	0.45	0.96	799	731	0.91
S165-F101-C	280.1	53.9	31.4	0.60	0.72	1167	1143	0.98
Mean								0.93
COV								0.04

Table 3.10. Capacity comparison for concrete-filled double-skin tubes (SS as the outer tube)

3.4.3 Capacity prediction of SWSSC-filled GFRP tubes

3.4.3.1 Existing model for FRP-confined concrete

Teng et al. (2009) proposed a design-oriented stress-strain model for FRP-confined concrete. This model has been widely adopted by other researchers. Teng's model consists of a parabolic portion followed by a linear portion. This model allows the effects of confinement stiffness and the jacket strain capacity to be separately reflected in both the axial strain and the compressive strength equations. It accounts for the effect of confinement stiffness explicitly instead of having it reflected only through the confinement ratio (Teng et al. 2009). More details of this model can be found in Teng et al. (2009).

A comparison between the stress-strain curves determined by Teng's model and test results of fully SWSSC-filled GFRP tube is shown in Fig. 3.21. The predicted ultimate stress, ultimate strain and the test results are also reported in Table 3.11, in which f_{cc} ' and ε_{cu} are the ultimate strength and strain of confined concrete, f_c ' and ε_{co} are the corresponding values of unconfined concrete. As shown in Fig. 3.21, there is obvious difference between the model prediction and test results: the stress-strain curves from tests are lower than that from Teng's model. Table 3.11 indicates that Teng's model overestimates the ultimate stress, whereas the ultimate strain prediction is reasonable. It should be noted that Teng's model was derived from FRP-confined concrete where all the FRP fibres were in the hoop direction, whereas in the current tests, 20%, 40%, and 40% fibres are in the angles of 15°, ±40° and ±75° with respect to longitudinal axis of GFRP tubes. It is worthwhile to mention that a similar study

recently carried out by Zhang et al. (2015) indicated that Teng's model can predict the stress-strain response of concrete confined by FRP tubes that had strength in both longitudinal and hoop directions. However, the ratio of hoop strength to longitudinal strength of FRP in Zhang et al.'s test (2015) $(f_{uh}/f_{ul}=6.8)$ is much higher than that in this research $(f_{uh}/f_{ul}=1.4)$.



Fig. 3.21. Comparison of stress-strain curves from Teng's model and test results

Table 3.11. Comparison of ultimate stress and strain from Teng's model and test results

Specimen	Teng's model		Test		Test/Teng's model	
	$f_{\rm cc}$ '/ $f_{\rm c}$ '	$\epsilon_{\rm cu}/\epsilon_{\rm co}$	$f_{\rm cc}$ '/ $f_{\rm c}$ '	$\epsilon_{cu}/\epsilon_{co}$	$f_{\rm cc}$ '/ $f_{\rm c}$ '	$\epsilon_{cu}/\epsilon_{co}$
F101-C	3.4	19.8	2.2	17.7	0.64	0.90
F165-C	2.4	14.1	1.8	14.0	0.77	1.00
Mean					0.71	0.95
COV					0.09	0.05

3.4.3.2 Proposed capacity formulae

As discussed in previous section, because the longitudinal strength of GFRP tube cannot be ignored, Teng's model is not appropriate to estimate the ultimate capacity of concrete-filled GFRP tubes. Therefore, it is necessary to propose a new method to determine the capacity of concrete-filled GFRP tubes.

The formulae to determine the capacity of concrete fully filled GFRP tubes are listed below:

$$N_{p} = f_{un} (A_{c} + A_{o} \frac{f_{ul}}{f_{uh}})$$
(3.13)

$$\frac{f_{un}}{f_c} = 1.12 + 2.64 \frac{f_I}{f_c}$$
(3.14)

$$f_l = \frac{2f_{uh}t_o}{D_o} \tag{3.15}$$
where A_c is the concrete area, A_o is the FRP tube cross-section area, f_{ul} is the longitudinal strength of FRP, f_{uh} is the hoop strength of FRP, f_c ' is the concrete strength, t_o is the thickness of outer FRP tube, D_o is the diameter of outer FRP tube, f_l is the confining pressure, and f_{un} is the nominal ultimate strength. The expression in Eq. (3.14) is determined by regression analysis of test data from Fam and Rizkalla's test (2001), Zhang et al.'s test (2015) and the current chapter (Fig. 3.22).



Fig. 3.22. Regression analysis of available data

A comparison between the predicted capacity and test capacity for all the existed data is summarised in Fig. 3.23 (the double-skin tubes in section 3.4.4 are also included). A reasonable agreement is found. Table 3.12 summarizes the comparison of capacity obtained by this new method and the current test results. The averaged ratio of test capacity to predicted capacity (N_p/N_t) ratio is 0.99 with coefficient of variation (COV) of 0.03.



Fig. 3.23. Comparison between predicted capacity and test capacity

1	Table 3.12.	Comparis	on between	new meth	od a	and test res	sults for f	ully filled	GFRP	tubes
	Specimen	f_{ul} (MPa)	$f_{\rm uh}$ (MPa)	<i>f</i> _c ' (MPa)	χ	f_l (MPa)	$N_{\rm t}$ (kN)	$N_{\rm p}$ (kN)	$N_{\rm p}/N_{\rm t}$	
	50-C	217.6	308.8	31.4	0	37.1	244	1.05	0.96	
	F101-C	217.6	308.8	31.4	0	19.3	670	0.98	1.03	
	F114-C	217.6	308.8	31.4	0	16.8	813	0.99	1.01	

F165-C	217.6	308.8	31.4	0 12.3	1336	0.97	1.03
Mean							0.99
COV							0.03

3.4.4 Capacity prediction of SWSSC-filled double-skin tubes (GFRP as the outer tube)

3.4.4.1 Existing model

Yu et al. (2010) applied Teng's model (2009) to concrete in hybrid FRP-concrete-steel double-skin tubular columns (referred as "modified Teng's model" in this chapter). Based on their research, the compressive strength of confined concrete is mainly dependent on the confinement stiffness and the FRP rupture strain but not the void ratio. They also found that the ultimate axial strain is related to void ratio and proposed a modified version to calculate the ultimate axial strain of the confined concrete.

A comparison between the stress-strain curves determined by the modified Teng's model and test results is shown in Fig. 3.24. The predicted ultimate stress, ultimate strain and the test results are also reported in Table 3.13. As shown in Fig. 3.24, similar to fully filled tubes, there is obvious difference between the model prediction and test results: the stress-strain curves from tests are lower than those from modified Teng's model. Table 3.13 indicates that Teng's model over-estimate the ultimate stress, whereas the ultimate strain prediction is reasonable. This is consistent with that observed for fully filled GFRP tubes described in Section 3.4.3.1 of this chapter.





The stand of an and show of the stand of the	Table 3.13. Com	parison of ultimate s	stress and strain fro	om Teng's model	and test results
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Sussimon	Teng's	model	Test		Test/Ter	Test/Teng's model		
specifien	$f_{\rm cc}$ '/ $f_{\rm c}$ '	$\epsilon_{cu}/\epsilon_{co}$	$f_{\rm cc}$ '/ $f_{\rm c}$ '	$\epsilon_{cu}/\epsilon_{co}$	$f_{\rm cc}$ '/ $f_{\rm c}$ '	$\epsilon_{cu}\!/\epsilon_{co}$		
F114-F50-C	2.9	19.0	2.3	18.8	0.81	0.98		
F165-F101-C	2.4	17.0	1.5	15.9	0.65	0.93		
F114-S50-C	2.8	18.7	2.1	16.9	0.75	0.90		
F165-S101-C	2.3	16.3	1.6	18.7	0.72	1.15		

Mean	0.73	0.99
COV	0.08	0.10

3.4.4.2 Proposed capacity formulae

For concrete-filled double-skin tubes, based on an earlier study (Yu et al. 2010) on concrete-filled double-skin FRP wraps, it is assumed that the capacity of double-skin tubes consists of the capacity of outer tube with sandwiched concrete and the capacity of inner tube. Therefore the capacity of SWSSC-filled double-skin GFRP tubes (GFRP as the outer tube) can be determined by:

$$N_p = N_{co} + N_i \tag{3.16}$$

$$N_i = f_{av,i} A_i \tag{3.17}$$

where N_{co} is the capacity of outer tube with sandwiched concrete, which can be determined by Eq. (3.15), N_i is the capacity of inner tube. Because of the inner tube is surrounded by concrete, SS inner tube can reach stresses higher than the yield stress ($f_{0.2}$) and GFRP inner tube can reach stresses higher than the yield stress ($f_{0.2}$) and GFRP inner tube can reach stresses higher than the residual stress (σ_{res}). For SS inner tube, an average stress between the yield stress ($f_{0.2}$) and the ultimate strength (f_u) is adopted, i.e. $f_{av,i}$ in Eq. (3.17) is taken as ($f_{0.2}+f_u$)/2. For GFRP inner tube, an average stress between the residual stress (σ_{res}) and the ultimate strength (f_{ul}) is adopted, i.e. $f_{av,i}$ in Eq. (3.17) is taken as ($\sigma_{res} + f_{ul}$)/2.

Table 3.14 summarizes the comparison of capacity obtained by this new method and test results in the present study. A reasonable agreement (within 7% on average) is achieved.

Specimen	Outer (MPa)		f (MD _a)	$f'(\mathbf{MD}_{\mathbf{a}})$		$f(\mathbf{MD}_{2})$	N. (kN)	N (kN)	NT /NT
specifien	$f_{ m ul}$	$f_{ m uh}$	J _{yi} (MPa)	J _c (MPa)	χ	Ji (MPa)	$N_{\rm t}$ (KIN)	$N_{\rm p}$ (KIN)	/v _p //v _t
F114-F50-C	217.6	308.8	164.6	31.4	0.45	15.7	795	688	0.87
F165-F101-C	217.6	308.8	135.8	31.4	0.63	12.2	880	893	1.01
F114-S50-C	217.6	308.8	462.5	31.4	0.42	15.6	872	814	0.93
F165-S101-C	217.6	308.8	485.8	31.4	0.64	11.4	1301	1160	0.89
Mean									0.93
COV									0.06

Table 3.14. Comparison between new method and test results for double-skin tubes (GFRP as the outer tube)

3.5 Conclusions

This chapter presents an experimental investigation on seawater and sea sand concrete (SWSSC) filled glass fibre reinforced polymer (GFRP) and stainless steel (SS) circular tubes. A total of 24 stub columns, including hollow SS or GFRP tubes, SWSSC fully filled tubes and double-skin tubes, were tested under axial compression with the load applied to concrete and tubes simultaneously. The following observations and conclusions are made based on limited experimental data.

1. The proposed SWSSC mix achieved a compressive strength ~ 31 MPa with desired workability for fully filled tubes and double skin tubes.

2. The ultimate hoop rupture strain of GFRP tube agrees well with that from disk-split test. The ratio of hoop strength to longitudinal strength of GFRP in this chapter is about 1.4.

3. The behaviour of SWSSC-fully filled SS tubes is similar to that of double skin tubes with SS as the outer tube. They are much more ductile than SWSSC-fully filled GFRP tubes and double skin tubes with GFRP as the outer tube.

4. The concrete enhancement caused by SS tube is slightly lower than that by GFRP tube, but the confinement lasts for larger axial strain. When the outer tube is GFRP the influence of inner tube type on concrete confinement is negligible. However, when the outer tube is stainless steel, the ultimate stress is similar, but the inner SS tube will result in a much more ductile behaviour than that with GFRP inner tube.

5. Capacity formulae were proposed to estimate the load carrying capacity of SWSSC fully filled SS or GFRP tubes, and that of double skin tubes with four combinations of inner and outer tubes, i.e. SS and SS, SS and GFRP, GFRP and GFRP and GFRP and SS. Reasonable agreement with experimental data has been achieved.

Research is being conducted on the durability of SWSSC-filled SS tubes and FRP tubes.

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References

AS 1141.11.1 (2009), Methods for sampling and testing aggregates – Particle size distribution – Sieving method, Standards Australia, Sydney.

AS 1391 (2007), Metallic materials-Tensile testing at ambient temperature, Standards Australia, Sydney.

AS/NZS 4673 (2001), Cold-formed stainless steel structures, Standards Australia, Sydney.

ASTMD 2290-12 (2012), Standard test method for apparent hoop tensile strength of plastic or reinforced plastic pipe, American Society for Testing and Materials, West Conshohocken, PA.

ASTM D3039/D3039M-14 (2014), Standard test method for tensile properties of polymer matrix composite materials, American Society for Testing and Materials, West Conshohocken, PA.

Chuah, S., Duan, W.H., Pan, Z., Korayem, A.H., Zhao, X.L., Collins, F. and Sanjayan, J.G. (2016), The properties of fly ash based geopolymer mortarsmade with dune sand, Materials and Design, 92, 571-578.

Fam, A.Z. and Rizkalla, S.H. (2001), Behavior of axial loaded concrete-filled circular fiber-reinforced polymer tubes, ACI Strucural Journal, 98(3), 280-289.

Gourley, B.C., Tort, C., Denavit, M.D., Schiller, P.H. and Hajjar, J.F. (2008), A synopsis of studies of the monotonic and cyclic behaviour of concrete-filled steel tube members, connections, and frames, Report No. NSEL-008, NSEL Report Series, Department of Civil and Environmental Engineering, University of Illinois at Urbana-Champaign, USA.

Han, L.H., Yao, G.H. and Zhao, X.L. (2005), Tests and calculations for hollow structural steel (HSS) stub columns filled with self-consolidating concrete (SCC), Journal of Constructional Steel Research, 61, 1241-1269.

Han, L.H., Ren, Q.X. and Li, W. (2011), Tests on stub stainless steel-concrete-carbon steel doubleskin tubular (DST) columns, Journal of Constructional Steel Research, 67, 437-452.

Han, L.H. and Yang, Y.F. (2007), Technology of modern concrete-filled steel structures, China Architecture & Building Press, Beijing, P.R. China.

Huang, H.X. (2007), An experimental study on the durability of sea sand concrete, Master thesis, Jinan University, Jinan, China (in Chinese).

Kaushik, S.K. and Islam, S. (1995), Suitability of sea water for mixing structural concrete exposed to a marine environment, Cement and Concrete Composites, 17(3), 177-185.

Kochhar, R. (2014), 10 projections for the global population in 2050, Pew Research Report, http://www.pewresearch.org/author/rkochhar/, 2014, (accessed 3 February 2014).

Kupwade-Patil, K. and Allouche, E. (2013), Impact of alkali silica reaction on fly ash-based Geopolymer concrete, Journal of Materials in Civil Engineering, 25(1), 131-139.

Lam, D. and Gardner, L. (2008), Structural design of stainless steel concrete filled columns, Journal of Constructional Steel Research, 64(11), 1275-1282.

Lam, L. and Teng, J.G. (2004), Ultimate condition of fiber reinforced polymer-confined concrete. Journal of Composites for Construction, 8(6), 539-548.

Lin, M.L. and Tsai, K.C. (2005), Behaviour of double-skinned composite steel tubular columns subjected to combined axial and flexural loads, In Proceedings of the 1st International Conference on Steel & Composite Structures, Pusan.

Mohammed, T.U.T., Hamada, H. and Yamaji, T. (2004), Performance of seawater-mixed concrete in the tidal environment, Cement and Concrete Research, 34(4), 593-601.

Nakanishi, K., Kitada, T. and Nakai, H. (1999), Experimental study on ultimate strength and ductility of concrete filled steel columns under strong earthquakes, Journal of Constructional Steel Research, 51(3), 297-319.

Nong, R. (2008), The experimental study on mechanical properties of sea sand concrete components, Master thesis, Harbin Institute of Technology, Harbin, China (in Chinese).

Ozbakkaloglu, T., Lim, J.C. and Vincent, T. (2013), FRP-confined concrete in circular sections: Review and assessment of stress–strain models, Engineering Structures, 49, 1068-1088.

Shams, M. and Saadeghvaziri, M.A. (1997), State of the art of concrete-filled steel tubular columns, ACI Structural Journal, 94(5), 558-571.

Shanmugam, N.E. and Lakshmi, B. (2001), State of the art report on steel-concrete composite columns, Journal of Constructional Steel Research, 57(10), 1041-1080.

Tam, V.W.Y., Wang, Z. and Tao, Z. (2014), Behaviour of recycled aggregate concrete filled stainless steel stub columns, Materials and Structures, 47(1-2), 293-310.

Tan, Z, Han, L.H. and Zhao, X.L. (2004), Behaviour of concrete-filled double skin (CHS inner and CHS outer) steel tubular stub columns and beam-columns, Journal of Constructional Steel Research, 60, 1129-1158.

Teng, J.G., Jang, T., Lam, L. and Luo, Y.Z. (2009), Refinement of a design-oriented stress–strain model for FRP-confined concrete, Journal of Composites for Construction, 13(4), 269-278.

Teng, J.G. and Lam, L. (2004), Behavior and modeling of fiber reinforced polymer-confined concrete, Journal of Structural Engineering, 130(11), 1713-1723.

Uy, B., Tao, Z. and Han, L.H. (2011), Behaviour of short and slender concrete-filled stainless steel tubular columns, Journal of Constructional Steel Research, 67(3), 360-378.

Wang, Y., Qian, X.D., Liew, J.Y.R. and Zhang, M.H. (2014), Experimental behavior of cement filled pipe-in-pipe composite structures under transverse impact, International Journal of Impact Engineering, 72, 1-16.

Yang, Y.F. and Ma, G.L. (2013), Experimental behaviour of recycled aggregate concrete filled stainless steel tube stub columns and beams, Thin-Walled Structures, 66, 62-75.

Yu, T., Teng, J.G. and Wong, Y.L. (2010), Stress-strain behavior of concrete in hybrid FRP-concretesteel double-skin tubular columns, Journal of Structural Engineering, 136(4), 379-389.

Zhang, B., Yu, T. and Teng, J.G. (2015), Behavior of concrete-filled FRP tubes under cyclic axial compression, Journal of Composites for Construction, 19(3), 04014060.

Zhao, X.L. and Han, L.H. (2006), Double skin composite construction, Progress in Structural Engineering and Materials, 8(3), 93-102.

Zhao, X.L., Han, L.H. and Lu, H. (2010), Concrete-filled tubular members and connections, CRC Press, London.

Chapter 4

Tests on seawater and sea sand concrete-filled CFRP, BFRP and stainless steel tubular stub columns

Abstract

This chapter presents an experimental study on concrete-filled circular tubes that consisted of seawater and sea sand concrete (SWSSC), stainless steel (SS) tube, carbon fibre reinforced polymer (CFRP) tube, and basalt fibre reinforced polymer (BFRP) tube. A total of 38 stub columns, which included 12 hollow section tubes, 12 fully SWSSC-filled tubes and 14 SWSSC-filled double-skin tubes, with four combinations of inner and outer tubes, were tested under axial compression. Tensile coupon tests and "disk-split" tests were conducted to obtain the material properties of SS, CFRP and BFRP. Ultimate strain of SWSSC-filled tubes and stress-strain curves of the confined concrete were characterised in the study. The effects of some key parameters (e.g., tube diameter-to-thickness ratio, cross-section types, outer tube types, and inner tube types) on the confinement effects were also discussed. Comparisons were made among CFRP, BFRP and glass fibre reinforced polymer (GFRP) in terms of confinement to SWSSC. The capacity prediction formulae previously proposed by the authors for SWSSC filled GFRP tubes were found to be reasonable for estimating the load carrying capacity of SWSSC filled CFRP and BFRP tubes.

Keywords

Seawater and sea sand concrete (SWSSC), CFRP, BFRP, GFRP, stainless steel, stub columns, axial compression

Nomenclature

- *A*_c Cross-section area of concrete
- Acn Nominal concrete area
- *A*_i Cross-section area of inner tube
- *A*_o Cross-section area of outer tube
- *A*_s Cross-section area of steel tube
- *D*_i Diameter of inner tube
- *D*_o Diameter of outer tube
- *E*_h Elastic modulus of FRP in hoop direction
- E_l Elastic modulus of FRP in longitudinal direction
- *E*_o Initial elastic modulus of stainless steel
- $f_{0.2}$ 0.2% proof stress of stainless steel
- $f_{\rm c}$ ' Concrete strength
- f_{cc} ' Confined concrete strength
- *f*_l Confining pressure
- fscy Nominal yielding strength of composite sections
- $f_{\rm u}$ Ultimate strength of stainless steel
- *f*_{uh} Ultimate strength in hoop direction of FRP (disk-split test)
- *f*_{ul} Ultimate strength in longitudinal direction of FRP (tensile coupon test)

- f_{un} Nominal ultimate strength of composite sections
- f_y Yielding strength (= $f_{0.2}$ for SS)
- f_{yi} Yielding strength of inner tube
- f_{yo} Yielding strength of outer tube
- *N*_p Predicted capacity
- Nt Test capacity
- *t*_i Thickness of inner tube
- *t*_o Thickness of outer tube
- ε_{cc} Ultimate strain of confined concrete
- ε_t Axial strain of hollow sections corresponding to the peak load
- ε_{th} Ultimate hoop strain of FRP tube
- ε_{tl} Ultimate axial strain
- ε_{uh} Ultimate strain of FRP in hoop direction (disk-split test)
- ε_{ul} Ultimate strain of FRP in longitudinal direction (tensile coupon test)
- χ Void ratio
- vh Poisson's ratio of FRP in hoop direction
- v_l Poisson's ratio of FRP in longitudinal direction
- ξ Confinement factor
- $\sigma_{\rm c}$ Stress in concrete
- $\sigma_{\rm res}$ Residual strength of FRP hollow sections
- $\sigma_{\rm t}$ Failure stress of hollow sections

4.1 Introduction

Concrete-filled tubes (CFTs) are being extensively used as the main structural members for resisting axial load in bridge piers and high-rise building columns. The CFT composed of the core concrete and encasing outer tubes is referred as fully concrete-filled tubes in this chapter (Zhao et al. 2010). In order to reduce the self-weight and increase the stiffness, concrete-filled double-skin tubes (CFDST), which consist of an outer and an inner tube with concrete-filled between them, were developed in recent decades (Zhao and Han 2006; Teng et al. 2007). As the core concrete is confined by the encasing tubes and the buckling of the tubes is delayed by the infilled concrete, CFTs and CFDST exhibit greater load-carrying capacities and ductility in comparison with unfilled tubes or plain concrete.

The seawater and sea sand concrete (SWSSC), that utilizes alkali-activated slag as binding material and sea sand as fine aggregate, was investigated in the present study with a view to replacing the conventional ordinary Portland cement (OPC)-based concrete that will avoid the consumption of fresh water, river sand and OPC. Existing literature (Kaushik and Islam 1995; Kupwade-Patil and Allouche 2013; Mohammed 2004) suggests that the use of geo-polymer (e.g. slag) can considerably reduce the expansion caused by alkali silica reaction (ASR), whereas the mechanical properties of

SWSSC is similar to those of conventional Portland concrete. However, the chloride ions in SWSSC can rapidly corrode the carbon steel. Therefore, the corrosion resistant materials (i.e., stainless steel and fibre reinforced polymer) have been investigated in this study.

Stainless steel (SS) has no obvious yield point with considerable strain hardening. Past researches (Lam and Gardner 2008; Uy et al. 2011) on fully concrete-filled stainless steel tubular columns have demonstrated their desirable structural performance (e.g. greater capacity and ductility) and indicated the current design method for concrete-filled carbon steel tubes to be conservative. However, very little studies are reported on concrete-filled double-skin SS tubes.

The increase demand of the use of fibre reinforced polymers (FRPs) in civil engineering applications is due to their favourable strength to weight ratio and high corrosion resistance. Among different types of FRPs, glass fibre reinforced polymers (GFRPs) are already widely used in civil engineering. It is well known that the carbon fibre reinforced polymers (CFRPs) have much higher strength and elastic modulus than GFRPs and the basalt fibre reinforced polymers (BFRPs) have similar mechanical properties to GFRPs for the similar fibre volume fractions (Lopresto et al. 2011; ACI440.2R-08 2008). Many studies (Teng and Lam 2004; Teng et al. 2009; Ozbakkaloglu et al. 2013) have been conducted on concrete-filled FRP wraps (with fibre exclusively oriented in hoop direction) as an important approach for strengthening existing structures. Prof. J.G. Teng of The Hong Kong Polytechnic University proposed the use of seawater and sea sand concrete with FRP to construct marine/coastal structures (Teng et al. 2011) and later promoted it internationally (Teng 2014). In recent years, researchers (Fam and Rizkalla 2001; Zhang et al. 2015) have started to apply concretefilled GFRP tubes (with glass fibres oriented both in hoop and longitudinal directions) in new constructions since the GFRP tube can be used as permanent formwork. The author has recently reported performance of SWSSC-filled GFRP tubes (Chapter 3, Li et al. 2016). To the best of authors' knowledge, there are no experimental studies on SWSSC-filled CFRP or BFRP tubes.

This chapter presents an experimental study on concrete-filled circular tubular stub columns made of seawater and sea sand concrete (SWSSC), stainless steel (SS) tube, carbon fibre reinforced polymer (CFRP) tube or basalt fibre reinforced polymer (BFRP) tube. Both the fully concrete-filled tubes and concrete-filled double-skin tubes were tested under axial compression. The SWSSC mixture developed by the authors (Chapter 3) was adopted in this testing program. The material properties of SWSSC, SS, CFRP and BFRP were obtained by standard testing methods. The load sharing of core concrete and encasing tubes as well as confinement effect provided by three types of FRPs were investigated. Finally, a unified approach was adopted to predict the load carrying capacity of SWSSC-filled SS, BFRP, CFRP and GFRP tubes in compression.

4.2 Experimental Program

4.2.1 Specimen

A total of 38 circular stub columns, including 12 hollow section tubes, 12 fully SWSSC-filled tubes, and 14 SWSSC-filled double-skin tubes, were prepared and tested in the present study. Four different tube dimensions (i.e. with nominal tube diameter of 50 mm, 101 mm, 114 mm and 165 mm, and nominal tube thickness of 3 mm) were selected. In order to eliminate the influence of global buckling and end effects, the length of fully SWSSC-filled tubes with outer dimeter of 50 mm (i.e. S50-H, C50-H, B50-H, S50-C, C50-C, and B50-C) was 150 mm and the length of all the other specimens was 400 mm.

The cross-section configuration of the specimens is illustrated in Fig. 4.1 and the measured dimensions of the specimens are listed in Table 4.1, in which the concrete strength (f_c ') and failure loads (N_i) are also included. The label of the specimen consists of the outer tube material ("S" for stainless steel, "C" for CFRP and "B" for BFRP) followed by outer tube nominal diameter ("50", "101", "114", and "165"), inner tube material followed by inner tube nominal diameter (if applicable), and cross-section type indicator ("H" for hollow sections and "C" for SWSSC-filled tubes). For example, S50-C refers to fully SWSSC-filled SS tubes with D_o of 50 mm, and S114-B50-C refers to SWSSC-filled double skin tubes with an SS outer tube ($D_o = 114$ mm) and BFRP inner tube ($D_i = 50$ mm). A similar labelling system was adopted in Chapter 3 where the letter "F" defined the GFRP filled tubes. However, in this current chapter more specific identifications have been used. Accordingly, letters "C", "B" and "G" have been incorporated into the labels to represent CFRP, BFRP and GFRP respectively. For example, specimen "S114-F50-C" is now called "S114-G50-C", which refers to SWSSC-filled double skin tubes with an SS outer tube ($D_o = 114$ mm) and GFRP inner tube ($D_i = 50$ mm).





<u>Care simon</u>	Outer	tube (m	m)	Inner tu	Inner tube (mm)			N_{t}
Specimen	$D_{ m o}$	to	Mat.	D_{i}	t _i	Mat.	(MPa)	(kN)
S50-Н	50.9	3.07	SS	N/A	N/A	N/A	N/A	200
S101-H	101.9	2.79	SS	N/A	N/A	N/A	N/A	250
S114-H	114.1	2.79	SS	N/A	N/A	N/A	N/A	332
S165-H	168.4	3.22	SS	N/A	N/A	N/A	N/A	536
С50-Н	50.5	2.81	CFRP	N/A	N/A	N/A	N/A	83
С101-Н	99.9	2.81	CFRP	N/A	N/A	N/A	N/A	145
С114-Н	114.6	2.75	CFRP	N/A	N/A	N/A	N/A	172
С165-Н	158.1	2.79	CFRP	N/A	N/A	N/A	N/A	203
В50-Н	50.0	2.71	BFRP	N/A	N/A	N/A	N/A	51
В101-Н	100.0	2.92	BFRP	N/A	N/A	N/A	N/A	94
B114-H	114.5	2.78	BFRP	N/A	N/A	N/A	N/A	88
B165-H	157.7	2.71	BFRP	N/A	N/A	N/A	N/A	124
S50-C	50.9	3.07	SS	N/A	N/A	N/A	35.8	235 ^a
S101-C	101.9	2.79	SS	N/A	N/A	N/A	35.8	570 ^a
S114-C	114.1	2.79	SS	N/A	N/A	N/A	35.8	766 ^a
S165-C	168.4	3.22	SS	N/A	N/A	N/A	35.8	1449 ^a
C50-C	50.5	2.81	CFRP	N/A	N/A	N/A	35.8	388
C101-C	99.9	2.81	CFRP	N/A	N/A	N/A	35.8	1131
C114-C	114.6	2.75	CFRP	N/A	N/A	N/A	35.8	1416
C165-C	158.1	2.79	CFRP	N/A	N/A	N/A	35.8	2372
В50-С	50.0	2.71	BFRP	N/A	N/A	N/A	32.8	259
B101-C	100.0	2.92	BFRP	N/A	N/A	N/A	32.8	656
B114-C	114.5	2.78	BFRP	N/A	N/A	N/A	32.8	825
B165-C	157.7	2.71	BFRP	N/A	N/A	N/A	32.8	1345
S114-S50-C	114.1	2.79	SS	50.9	3.07	SS	39.4	852 ^a
S165-S101-C	168.4	3.22	SS	101.9	2.79	SS	39.4	1314 ^a
S114-C50-C	114.1	2.79	SS	50.5	2.81	CFRP	39.4	735 ^a
S165-C101-C	168.4	3.22	SS	99.9	2.81	CFRP	39.4	1271 ^a
C114-S50-C	114.6	2.75	CFRP	50.9	3.07	SS	39.4	1375
C165-S101-C	158.1	2.79	CFRP	101.9	2.79	SS	39.4	1698
С114-С50-С	114.6	2.75	CFRP	50.5	2.81	CFRP	39.4	1175
C165-C101-C	158.1	2.79	CFRP	99.9	2.81	CFRP	39.4	1219
S114-B50-C	114.1	2.79	SS	50.0	2.71	BFRP	32.8	676 ^a
S165-B101-C	168.4	3.22	SS	100.0	2.92	BFRP	32.8	1145 ^a
В114-S50-С	114.5	2.78	BFRP	50.9	3.07	SS	32.8	884
B165-S101-C	157.7	2.71	BFRP	101.9	2.79	SS	32.8	1053
B114-B50-C	114.5	2.78	BFRP	50.0	2.71	BFRP	32.8	651
B165-B101-C	157.7	2.71	BFRP	100.0	2.92	BFRP	32.8	703

Table 4.1. Details of specimens

^a: The load corresponding to 5% axial strain was adopted as N_t

4.2.2 Material properties

4.2.2.1 Seawater and sea sand concrete (SWSSC)

A proper SWSSC mix was developed by the authors (Chapter 3) to achieve the target strength and desirable workability with the slump of fresh concrete reaching 160 mm. The same concrete mix developed in Chapter 3 was adopted in this study. The mix includes: slag (360 kg/m³), seawater (190 kg/m³), sea sand (830 kg/m³), coarse aggregate (1130 kg/m³), hydrate lime slurry (14.4 kg/m³) and sodium metasilcate (38.4 kg/m³) as activator. More details of the SWSSC can be found in Chapter 3.

Three batches of concrete were cast and three identical standard cylinders (with diameter of 100 mm and height of 200 mm) were prepared for each batch in order to measure the concrete strength (f_c '). All the specimens and cylinders were sealed by plastic film to avoid the moisture evaporation and aged to 28 days. The concrete strength (f_c ') is listed in Table 4.1, which varies from 32.8 MPa to 39.4 MPa.

4.2.2.2 Stainless steel

The SS tubes were made of 316 grade austenitic stainless steel in accordance with AS/NZS 4673 (2001). Three tensile coupons were cut from each size of SS tube and the ends of the coupons were flattened in order to be gripped by test machine. The tensile coupon test was conducted according to AS 1391 (2007) with a loading rate of 1 mm/min. The averaged proof stress ($f_{0.2}$) and ultimate tensile strength (f_u) are summarised in Table 4.2. A typical stress-strain curve for stainless steel is plotted in Fig. 4.2, in which the full range strain was obtained from laser-extensometer (gage length = 50 mm), the initial range strain was obtained from strain gauges attached at the middle part of coupons and the initial elastic modulus (E_o) is 195 GPa.



Fig. 4.2. Typical stress-strain curve of stainless steel

Tube size	$f_{0.2}$ (MPa)	$f_{\rm u}$ (MPa)
S50	228.2	562.1
S101	225.7	656.4
S114	280.7	617.8
S165	281.1	615.8
Mean	253.9	613.0
COV	0.11	0.05

Table 4.2. Material properties of stainless steel

4.2.2.3 Carbon fibre reinforced polymer (CFRP) and basalt fibre reinforced polymer (BFRP)

The CFRP and BFRP tubes were fabricated by filament winding process with epoxy as the matrix. Based on the data provided by the manufacture, the fibre-volume fraction is about 60%. In order to provide strength and stiffness in both longitudinal and hoop directions, the fibres were oriented in different directions and 20%, 40%, and 40% fibres were in the angle of 15° , $\pm 40^{\circ}$, and $\pm 75^{\circ}$ with respect to the longitudinal axis of tubes. The same orientation was used for manufacturing GFRP tubes reported in Chapter 3.

The material properties of CFRP and BFRP in the longitudinal direction were obtained by tensile coupon test. Three coupons were cut from each size of tubes. Two pairs of grippings were attached on the coupon ends to ensure proper gripping. The tensile test was conducted in accordance with ASTM D3039 (2014) with a loading rate of 0.5 mm/min. The material properties in the hoop direction were obtained by the "disk-split" test specified in ASTM D2290 (2012) with a loading rate of 0.5 mm/min. Three 20 mm wide rings were cut from each size of tubes for "disk-split" test. Details of the test setup can be found in Chapter 3.

A summary of the test results is reported in Table 4.3, in which f_{ul} , ε_{ul} , E_{l} , and v_{l} is the ultimate strength, ultimate strength, and Poisson's ratio in longitudinal direction, whereas f_{uh} , ε_{uh} , E_{h} , and v_{h} is the ultimate strength, ultimate strain, elastic modulus, and Poisson's ratio in hoop direction. The strain was the averaged strain gauge readings and the elastic moduli were calculated in accordance with ASTM D3039 (2014). The data for GFRP (with similar fibre volume fraction and fibre orientations) in Chapter 3 is also listed in Table 4.3.

Since the same manufacture process, fibre volume fraction and fibre orientations were used for different tube sizes, there is no significant difference in material properties between tubes with different diameters, which is also demonstrated in Table 4.3. In the following discussion, the influence of tube diameter is ignored and material properties of CFRP or GFRP is taken as the averaged value of all the tubes. As shown in Table 4.3, the ultimate strength and elastic moduli in

hoop direction are much higher than those in the longitudinal direction. The f_{uh} to f_{ul} ratio (f_{uh}/f_{ul}) is 2.4, 2.7, and 1.4 for CFRP, BFRP and GFRP respectively. CFRP has the highest ultimate strength and elastic moduli in both directions, which is in agreement with previous studies (ACI440.2R-08; Chen et al. 2008). BFRP has the lowest ultimate strength and elastic modulus in longitudinal direction, while the material properties of BFRP in hoop direction are similar to those of GFRP. With regard to ultimate strain, there is no obvious difference between ultimate strain in longitudinal and hoop directions. The ultimate strain of CFRP is much less than that of BFRP and GFRP. The Poisson's ratio for all the FRPs is about 0.3 except for CFRP in hoop direction, which indicates that the Poisson's ratio does not vary considerably for different type of fibres.

FRP type Tu C5 C1 C1 C1 C1 C1 C1 C1 C1 C1 C1 C1 C1 C1	Tube size	Longitudi	nal directi	on		Hoop direction			
	Tube size	$f_{\rm ul}$ (MPa)	$\mathcal{E}_{\mathrm{ul}}$	E _l (GPa)	v_l^{a}	$f_{\rm uh}$ (MPa)	$\mathcal{E}_{\mathrm{uh}}$	<i>E</i> _h (GPa)	$v_{\rm h}$
	C50	244.7	0.0089	40.6	0.25	631.4	0.0096	75.6	0.51
	C101	240.5	0.0092	37.8	0.27	581.8	0.0102	65.1	0.51
CEDD	C114	230.3	0.0086	39.6	0.27	562.6	0.0103	63.6	N/A
CFRP	C165	256.0	0.0084	44.0	0.27	595.5	0.0099	62.5	0.53
	Mean	242.9	0.0088	40.5	0.26	592.8	0.0100	66.7	0.52
	COV	0.044	0.040	0.064	0.038	0.049	0.032	0.090	0.022
	B50	116.0	0.0129	13.7	0.28	334.6	0.0145	22.8	0.30
	B101	128.3	0.0154	12.4	0.30	329.4	0.0147	24.1	N/A
DEDD	B114	126.0	0.0138	12.1	0.29	340.6	0.0158	25.8	N/A
BFKP	B165	123.1	0.0142	12.7	0.29	319.6	0.0144	24.4	0.30
	Mean	124.0	0.0142	12.7	0.29	331.1	0.0149	24.3	0.30
	COV	0.043	0.074	0.055	0.028	0.027	0.043	0.051	0.000
CEDD	Mean	217.6	0.0190	20.1	0.32	308.8	0.0139	25.2	N/A
GFRP [♭]	COV	0.11	0.31	0.10	0.08	0.16	0.07	0.04	N/A

Table 4.3. Material properties of CFRP, BFRP and GFRP

^a: Poisson's ratio in longitudinal direction was obtained from stress-strain curves of hollow sections; ^b: From Chapter 3



Fig. 4.3. Typical stress-strain curves of CFRP, BFRP and GFRP (the data of GFRP is from Chapter 3)

Typical stress-strain curves of CFRP, BFRP and GFRP are plotted in Fig. 4.3. As shown in Fig. 4.3, the stress-strain behaviour of FRP in the hoop direction can be regarded as linear. However, the stress-strain behaviour of FRP in the longitudinal direction is not purely linear and some nonlinearity is observed during the stage before failure.

4.2.3 Test setup

All hollow section tubes were tested using a 500 kN capacity Baldwin machine, whereas all the SWSSC-filled tubes were tested in a 5000 kN capacity Amsler machine. The load was directly applied on the specimen through a loading plate. The cement paste was used to fill the gap caused by the shrinkage of concrete so that the axial load could be simultaneously applied on the core concrete and encasing tubes.

Three linear variable displacement transducers (LVDTs) were placed in an equidistant configuration around the specimens to measure the axial end shortening. Three stain gauges were fixed in the longitudinal direction and another three strain gauges were fixed in the circumferential direction at the mid-height of specimens. More details of the test setup and instrumentations can be found in Chapter 3.

The axial strain can be obtained by two approaches: (1) averaged axial end shortening divided by specimen length which is called "axial strain (from LVDT)"; (2) averaged reading of strain gauges in longitudinal direction which is called "axial strain (from strain gauge)". In the following discussion, if without specific noting, the axial strain refers to "axial strain (from LVDT)".

4.3 Test Results

4.3.1 Hollow sections

As the length-to-diameter ratio is less than 4 for all the SS hollow section tubes, the specimens failed by yielding and plastic local buckling. The failure modes of the specimens are shown in Fig. 4.4 where the elephant foot was formed during the late stage of the test. The test results for SS hollow section tubes under axial compression are summarized in Table 4.4, in which $f_{0.2}$ is 0.2% proof stress of SS, σ_t is the failure stress (equals to the peak load divided by cross-section area), ε_t is the axial strain corresponding to the peak load. As shown in Table 4.4, with the increase of D_0/t_0 , the $\sigma_t/f_{0.2}$ and ε_t decrease since the cross-section becomes more slender. The stress-strain curves of SS hollow sections are plotted in Fig. 4.5, where obvious strain hardening can be observed. A comparison between Fig. 4.5 (a) and (b) indicates that when the tube is under large deformation (i.e. after the peak load), the strain obtained from LVDT cannot represent the real strain of the SS tube.



Fig. 4.4. Failure modes of SS hollow sections



Fig. 4.5. Stress-strain curves of SS hollow sections

Specimen	$D_{\rm o}/t_{\rm o}$	f _{0.2} (MPa)	$\sigma_{\rm t}$ (MPa)	$\sigma/f_{0,0}$	<i>E</i> t		
				00/10.2	From LVDT	From strain gauge	
S50-H	16.6	228.2	433.3	1.90	0.1081	0.0722	
S101-H	36.6	225.7	288.1	1.28	0.0306	0.0295	
S114-H	40.9	280.7	340.5	1.21	0.0241	0.0179	
S165-H	52.3	281.1	320.3	1.14	0.0156	0.0124	

Table 4.4. Test results of SS hollow section	ıs
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The failure mode of CFRP and BFRP hollow section tubes is local buckling near the ends. The stressstrain curves are plotted in Fig. 4.6, and a summary of the test results is listed in Table 4.5. The hollow sections can resist the load linearly until a sudden failure causing a dramatic drop of applied load. Because of the occurrence of local buckling, the compression strength (σ_t) and ultimate strain (ε_t) of hollow tubes generally cannot reach the tensile strength (f_{ul}) and ultimate tensile strain (ε_{ul}) obtained by coupon test. With the increase of D_o/t_o , the σ_t/f_{ul} and ε_t generally decrease slightly but this trend is not as clear as that for SS hollow sections. One possible reason is that the local buckling of FRP is more complicated due to the large number of fibres involved. As shown in Fig. 4.6, there is no nonlinearity of stress-strain curves for FRP hollow sections under axial compression, which is different from the results of tensile coupon test. The "stiffness" in Table 4.5 is the slope of stressstrain curves (before reaching the peak load) in Fig. 4.6. The stiffness (from strain gauges) for CFRP is close to the elastic modulus (E_l), which is as expected. However, the stiffness (from strain gauges) for BFRP is much higher than the E_l . After the occurrence of local buckling, the FRP hollow section still has some residual strength. In this chapter, the residual strength (σ_{res}) is roughly taken as the averaged stress after reaching the peak load, as defined later in Fig. 4.9(b).



(b) Axial strain obtained from strain gauge Fig. 4.6. Stress-strain curves of CFRP and BFRP hollow section

Specimen	D /t	Material properties			σ (MDa)	- /£	- (MD ₂)	£t		Stiffness (GPa)	
Specimen	D_0/l_0	f _{ul} (MPa)	$\mathcal{E}_{\mathrm{ul}}$	<i>E</i> ₁ (GPa)	o_t (MPa)	$o_{t/Jul}$	$o_{\rm res}$ (MPa)	From LVDT	From strain gauge	From LVDT	From strain gauge
С50-Н	18.0	242.9	0.0088	40.5	197.4	0.81	134.5	0.0071	0.0051	27.9	39.2
C101-H	35.6	242.9	0.0088	40.5	169.0	0.70	112.9	0.0078	0.0045	30.2	38.1
C114-H	41.7	242.9	0.0088	40.5	178.4	0.73	30.5	0.0074	0.0045	32.0	39.9
С165-Н	56.7	242.9	0.0088	40.5	149.6	0.62	76.1	0.0043	0.0034	35.4	42.9
Mean							88.5			31.4	40.0
COV							0.515			0.101	0.051
В50-Н	18.5	124.0	0.0142	12.7	126.5	1.02	43.2	0.0074	0.0031	16.8	24.2
B101-H	34.3	124.0	0.0142	12.7	105.8	0.85	58.8	0.0053	0.0023	20.6	25.7
B114-H	41.2	124.0	0.0142	12.7	90.2	0.73	69.5	0.0051	0.0027	19.6	22.6
B165-H	58.2	124.0	0.0142	12.7	94.0	0.76	65.5	0.0078	0.0026	18.1	23.0
Mean							59.3			18.8	23.9
COV							0.195			0.089	0.058

Table 4.5. Test results of CFRP and BFRP hollow sections

4.3.2 SWSSC-filled tubes

The load-axial strain curves of SWSSC-filled tubes are classified into three groups (Fig. 4.7) based on the outer tube material type: (a) SWSSC-filled SS tubes; (b) SWSSC-filled CFRP tubes; (c) SWSSC-filled BFRP tubes. In general, the material type of the outer tube has a greater influence on the behaviour. The strength and ductility of the concrete-filled tubes are substantially enhanced in comparison with the hollow tubes or the plain concrete.



Fig. 4.7. Load-axial strain curves of SWSSC-filled tubes

The tests for SWSSC-filled SS tubes were terminated due to the limited stroke of test machine (about 60 mm), and the ultimate capacity (N_t) was taken as the maximum load within 5% axial strain. Similar definition of N_t for concrete-filled SS tubes was also adopted by Lam and Gardner (2008). The ultimate capacity (N_t) for SWSSC-filled CFRP and BFRP tubes is taken as the maximum load the specimen can sustain during the test. The ultimate capacity of all the specimens is listed in Table 4.1.

Obvious strain hardening behaviour is observed for fully SWSSC-filled SS tubes (Fig. 4.8 (a)). When SS is used as both outer and inner tubes (Fig. 4.8(b)), the shape of load-axial strain curves of specimen S114-S50-C is similar to that of S165-S114-C but there is an obvious drop of applied load for specimen S165-S101-C with a more slender inner tube. For double-skin tubes with SS as the outer

tube and CFRP/BFRP as the inner tube (Fig. 4.8(c)), there is a sudden drop of applied load caused by the buckling of inner FRP tube that was accompanied by a loud noise. Fig. 4.8(b, c) indicates that the amplitude of the load drop mainly depends on the diameter-to-thickness ratio (D_0/t_0), void ratio (D_i/D_0) and inner tube type. In general, the SWSSC-filled SS tubes (both fully filled and double-skin tubes) exhibit high ductility. The failure mode was a folding failure mechanism (yielding and local buckling), which is in agreement with the observation of other researchers (Lam and Gardner 2008; Uy et al. 2011).







(g) Double-skin tubes (BFRP as the outer tube) Fig. 4.8. Load-axial strain curves and failure modes

Obvious bilinear response was observed for SWSSC-filled CFRP and BFRP tubes (Fig. 4.8(d-g)), which is different from the load-strain curves of SWSSC-filled SS tubes. The failure mode for SWSSC-filled FRP tube is the outer tube rupture in hoop direction (except specimen C165-C101-C, B114-B50-C, and B165-B101-C). Based on the test observation, the ruptures of the CFRP tubes were more sudden than those of the BFRP tubes. For specimens C165-C101-C, B114-B50-C, and B165-B101-C, local buckling of inner tubes occurred. It seems that slender CFRP and BFRP inner tubes are more susceptible to local buckling. These specimens failed in longitudinal direction (i.e. buckling of tubes and crushing of concrete) before the rupture of the outer tubes. Although the failure mode of

these specimens is different from the rest of the SWSSC-filled FRP tubes, their axial strain at failure (around 0.03 to 0.04) is very close to that for other specimens. As shown in Fig. 4.8(d-g), slight load drop was observed for the SWSSC-filled FRP specimens before reaching the ultimate capacity, which is caused by the local buckling of the FRP tubes. A comparison between Fig. 4.8(d) and (f) indicates that the strength enhancement caused by the CFRP tube is more considerable than the BFRP tube since the hoop strength of the CFRPs is much higher than that of the BFRPs. On the condition that the failure mode is tube rupture, there is not much difference of the shape of load-axial strain curves for fully filled tubes and double-skin tubes.

4.4 Discussion

4.4.1 Ultimate strain of CFRP and BFRP tubes

The first buckling strain refers to the axial strain corresponding to the first occurrence of local buckling for the CFRP and BFRP tubes, which can be observed during the test. The first buckling strain for tested specimens is summarized in Table 4.6, in which "Hollow" refers to the buckling strain of corresponding hollow sections.

	From I	LVDT	From strain gauge						
Specimen	Inner	Hollow	Inner/ Hollow	Outer	Hollow	Outer/ Hollow	Outer	Hollow	Outer/ Hollow
С50-С	N/A	N/A	N/A	0.010	0.007	1.35	0.009	0.005	1.69
C101-C	N/A	N/A	N/A	0.011	0.008	1.48	0.008	0.004	1.83
C114-C	N/A	N/A	N/A	0.009	0.007	1.25	0.007	0.004	1.50
C165-C	N/A	N/A	N/A	0.012	0.004	2.72	0.007	0.003	2.22
S114-C50-C	0.010	0.007	1.43	N/A	N/A	N/A	N/A	N/A	N/A
S165-C101-C	0.011	0.008	1.41	N/A	N/A	N/A	N/A	N/A	N/A
C114-S50-C	N/A	N/A	N/A	0.008	0.007	1.13	0.007	0.004	1.52
C165-S101-C	N/A	N/A	N/A	0.010	0.004	2.30	0.006	0.003	1.65
С114-С50-С	N/A	N/A	N/A	0.009	0.007	1.17	0.006	0.004	1.41
C165-C101-C	0.011	0.008	1.37	0.008	0.004	1.77	0.006	0.003	1.80
Mean	0.011			0.010			0.007		
В50-С	N/A	N/A	N/A	0.014	0.007	1.87	0.009	0.003	2.90
B101-C	N/A	N/A	N/A	0.008	0.005	1.59	0.007	0.002	3.22
B114-C	N/A	N/A	N/A	0.010	0.005	1.96	0.008	0.003	2.93
B165-C	N/A	N/A	N/A	0.010	0.008	1.34	0.008	0.003	2.93
S114-B50-C	0.007	0.007	1.00	N/A	N/A	N/A	N/A	N/A	N/A
S165-B101-C	0.008	0.005	1.42	N/A	N/A	N/A	N/A	N/A	N/A
B114-S50-C	N/A	N/A	N/A	0.010	0.005	1.94	0.008	0.003	2.91
B165-S101-C	N/A	N/A	N/A	0.011	0.008	1.35	0.008	0.003	3.21
В114-В50-С	0.008	0.007	1.05	0.009	0.005	1.80	0.006	0.003	2.39
B165-B101-C	0.006	0.005	1.20	0.011	0.008	1.36	0.007	0.003	2.76
Mean	0.007			0.010			0.008		

Table 4.6. First buckling strain of CFRP and BFRP tubes

Based on Table 4.6, several conclusions can be made: (1) the strain obtained by LVDTs is larger than that by strain gauges and similar findings were reported by other researchers (Chapter 3; Ozbakkaloglu and Xie 2016); (2) the tube diameter-to-thickness ratio does not substantially affect the first buckling strain, which is different from the case of hollow sections; (3) the first buckling strain is larger than the buckling strain of corresponding hollow sections, indicating the fill-in concrete can delay the buckling of FRP tube; (4) the first buckling strain of other tubes (from strain gauges) is slightly less than the ultimate tensile strain obtain from coupon test (ε_{ul} =0.0088 for CFRP, ε_{ul} =0.0142 for BFRP) due to the local buckling; (5) The first buckling strain of inner tubes is in the same range (0.006 to 0.01) for all three FRPs.

The ultimate axial strains (ε_{tl} , from LVDTs) corresponding to ultimate capacity are listed in Table 4.7, in which the ultimate tensile strain in longitudinal direction (ε_{ul}) is also provided. The specimens that failed by tube rupture have much higher ultimate axial strain than specimens that failed by tube buckling and concrete crushing (i.e. C165-C101-C, B114-B50-C, and B165-B101-C). Table 4.7 indicates that with the increase of outer tube diameter-to-thickness ratio (D_0/t_0), the ultimate axial strain generally decreases slightly. The ultimate axial strain is much lower than the material property obtained from tensile coupon test (i.e. ε_{ul}). Even though the tubes failed in the longitudinal direction, the specimen can still increasingly sustain the load as the core concrete is effectively confined by the encasing tubes in hoop direction.

Spacimon	Axial d	irection	Hoop direction			
Specifien	\mathcal{E}_{tl}	$\mathcal{E}_{\mathrm{ul}}$	$\mathcal{E}_{\mathrm{th}}$	$\mathcal{E}_{\mathrm{uh}}$		
С50-С	0.058		N/A			
C101-C	0.051		0.011	0.010		
C114-C	0.051		0.013			
C165-C	0.046	0.000	0.012			
C114-S50-C	0.052	0.009	0.012			
C165-S101-C	0.058		0.015			
C114-C50-C	0.056		0.014			
C165-C101-C ^a	0.028		0.007			
В50-С	0.053		N/A			
B101-C	0.038		0.014			
B114-C	0.032		0.014			
B165-C	0.034	0.014	0.018	0.015		
В114-S50-С	0.040	0.014	0.016	0.015		
B165-S101-C	0.035		0.016			
B114-B50-C ^a	0.034		0.011			
B165-B101-C ^a	0.024		0.007			

Table 4.7. Ultimate axial strain (from LVDTs) and ultimate hoop strain corresponding to ultimate capacity

The ultimate hoop strain (ε_{th}) is taken as the averaged readings of strain gauges in hoop direction and the results are summarized in Table 4.7. As mentioned previously, some double-skin specimens (i.e. C165-C101-C, B114-B50-C, and B165-B101-C) did not fail by tube rupture and the ultimate axial strain is much less than for the other specimens failed by tube rupture. The ultimate hoop strain of concrete-filled CFRP tubes (except specimens without tube rupture) is slightly higher than that obtained from "disk-split" test (ε_{uh}). The ε_{th} of BFRP tubes agrees well with the ε_{uh} , which is similar to the conclusions of GFRP tubes (Chapter 3). As shown in Table 4.7, the outer tube diameter-to-thickness ratio does not obviously affect the hoop rupture strain, and there is no obvious difference in hoop rupture strain between fully filled tubes and double-skin tubes.

4.4.2 Load distribution

It is well known that the confined concrete strength (f_{cc} ') is higher than the unconfined concrete strength (f_{c} ') due to the confinement effect provided by the encasing tubes. In order to discuss the confinement effect, the load resisted by core concrete should be firstly determined.



Fig. 4.9. Simplified stress-strain models

As both the core concrete and encasing tubes sustain the applied load simultaneously, the load shared by core concrete is calculated by the following criteria: (1) the load carried by concrete is equal to the difference between the applied load and the load carried by tubes at the same axial strain; (2) the load carried by SS tubes can be determined by the simplified model in Fig. 4.9(a), in which the axial loadstrain curves for corresponding hollow sections were obtained from the tests; (3) the load carried by outer CFRP and BFRP tube is the product of the axial strain (from strain gauges), stiffness (obtained from compression test on hollow sections: 40.0 GPa for CFRP tube and 23.9 GPa for BFRP tube) and cross-section area; (4) the load carried by inner CFRP and BFRP tube is determined by the simplified model in Fig. 4.9(b), in which the axial strain is obtained from LVDTs, *E* is elastic modulus of hollow sections (31.4 GPa for CFRP tube and 18.8 GPa for BFRP tube), f_{ul} is longitudinal strength, ε_l is the first buckling strain (as highlighted in load-strain curves in Fig. 4.8), σ_{res} is the averaged residual strength (listed in Table 4.5); (5) the stress in CFRP and BFRP tubes should not exceed the f_{ul} obtained by tensile coupon test. The criteria in the present chapter is similar to the method adopted in Chapter 3 except the addition of criterion (5) as an improvement of the method in Chapter 3. The load-distribution curves of the specimens are presented in Fig. 4.10. The stress in the concrete is estimated as the load carried by concrete divided by cross-section area of concrete. Although the stress-strain curves of concrete are not accurate due to the complexity caused by tube buckling, the relative comparison between specimens is still helpful in understanding the influence of some key parameter on the behaviour of confined concrete as discussed in the followings.















4.4.3 Concrete confinement

4.4.3.1 Effects of tube diameter-to-thickness ratio (D_o/t_o)

As shown in Fig. 4.11(a), the tube diameter-to-thickness ratio does not significantly affect the σ_c/f_c 'strain curves of fully SWSSC-filled SS tubes. Obvious bilinear stress-strain response is observed for concrete confined by CFRP and BFRP tubes (Fig. 4.11(b, c)). With the increase of D_o/t_o , both the confined concrete strength (f_{cc} ') and ultimate strain (ε_{cc}) decrease. This is in agreement with existing research on concrete confined by FRP wraps (Teng and Lam 2004), i.e. the increase of D_o/t_o can attribute to a decrease of confining pressure (f_1) on the concrete. A comparison between Fig. 4.11(b) and (c) indicates that CFRP tubes. Fig. 4.11(d) compares three types of FRP tubes fully filled with SWSSC. It seems that GFRP and BFRP provide similar confinement, whereas more confinement is achieved by CFRP.



4.4.3.2 Effects of cross-section types (fully filled and double-skin)

As shown in Fig. 4.12(a), compared to fully filled tubes, when the specimen is under large deformation, there is a drop of σ_c/f_c ' for double-skin tubes as the inner tube cannot effectively resist the inward expansion of concrete. The amplitude of the load drop and residual strength depend on the void ratio (D_i/D_o), outer tube diameter-to-thickness ratio (D_o/t_o), and inner tube material type. The load drop of double-skin tubes with SS as the outer tube and CFRP/BFRP as the inner tube is more severe due to the buckling of inner FRP tube.







Fig. 4.12(b) to Fig. 4.12(d) indicate that there is not much difference of σ_c/f_c '-strain curves for doubleskin tubes (CFRP or BFRP or GFRP as the outer tube and SS as the inner tube) and corresponding fully filled tubes. The rupture of the outer tube occurs earlier than the occurrence of large deformation of inner SS tube so that the inner tube can effectively resist the inward expansion of concrete. For double skin tubes with FRP as both outer and inner tubes, the behaviour is similar until the slender inner tube (C101 and B101) buckles at an axial strain around 0.03.

4.4.3.3 Effects of outer tube types

The types of outer tube (SS tube or FRP tube) can obviously affect the shape of σ_c/f_c '-strain curves (Fig. 4.13). The strength enhancement caused by FRP tube is more significant than that by SS tube, but the strain in SWSSC-filled SS tube is much higher than that in SWSSC-filled FRP tube. Furthermore, the confinement effect by CFRP is greater than that by BFRP or GFRP due to the higher hoop strength of the former (CFRP).



The different shapes of σ_c/f_c '-strain curves for SWSSC-filled SS tubes and SWSSC-filled FRP tubes are substantially attributed by their different material properties. When the lateral expansion of concrete exceeds that of FRP tube, the confining pressure on the concrete increase continually due to the linear stress-strain response of FRP in hoop direction. This kind of confinement effect is called as "passive confinement effect". However, "active confinement effect" occurs in SWSSC-filed SS tubes where confining pressure provided by SS tube slows down after the SS reaches yield strength (0.2 % proof strength).

4.4.3.4 Effects of inner tube types

In general, the buckling of inner FRP tube can cause a sudden drop of applied load. When the outer tube is SS (Fig. 4.14(a)), the influence of inner tube types on the shape of σ_c/f_c '-strain curves is not significant. The residual strength mainly depends on the void ratio (D_i/D_o) and the outer tube diameter-to-thickness ratio (D_o/t_o) . It is believed that when the specimen is under large deformation, the inner tube cannot restrain the inward expansion of core concrete.



For specimens with CFRP and BFRP as the outer tube, when the failure mode is tube rupture, the effects of inner tube types can be ignored. However, the inner FRP tubes in some specimens (e.g., C165-C101-C and B165-B101-C with slender inner tubes) cannot effectively restrain the concrete after axial strain reaches around 0.03 and the rupture of the outer FRP tube does not happen. In this case, the concrete strength enhancement caused by confinement effect and the ductility are much less than those of the corresponding double-skin tubes with SS as the inner tube since the hoop strength of the outer tube is not fully utilized. Fig. 4.14(d) compares the confinement provided by three types of FRPs. When SS inner tube is used, BFRP and GFRP outer tubes perform at a similar level, but CFRP outer tube provides greater confinement effect. When FRP is used as both inner and outer tubes, BFRP and GFRP have similar behaviour, whereas CFRP achieves greater confinement effect.

4.5 Capacity Prediction

4.5.1 Fully SWSSC-filled stainless steel tube

Current design codes do not cover the ultimate capacity of fully concrete-filled SS tubes under axial compression. Past research (Lam and Gardner 2008) indicated that the design methods for fully concrete-filled carbon steel tubes is conservative for fully concrete-filled SS tubes. Chapter 3 made some modifications of a design method for concrete-filled carbon steel tubes, which was proposed by Han et al. (2005), and applied it to fully SWSSC-filled SS tubes. The method proposed by Chapter 3 is adopted to estimate the load carrying capacity of specimens tested in the present study.

The predicted ultimate capacity (N_p) of fully SWSSC-filled SS tubes can be determined by Eqs. (4.1a) to (4.1c):

$$N_{\rm p} = (A_{\rm s} + A_{\rm c})f_{\rm scy} \tag{4.1a}$$

$$f_{\rm scy} = (1.14 + 1.4\xi) f_{\rm c}' \tag{4.1b}$$

$$\xi = \frac{A_{\rm s}f_{\rm y}}{A_{\rm c}f_{\rm c}'} \tag{4.1c}$$

where A_s is cross-section area of SS tube, A_c is cross-section area of concrete, f_c ' is concrete strength, f_y is yield strength (= $f_{0.2}$ for SS), ξ is confinement factor, and f_{scy} is nominal yield strength of composite sections. In Chapter 3, the relationship between f_{scy}/f_c ' and ξ (i.e. Eq. (4.1b)) was determined by the regression analysis of available existing data (Chapter 3; Lam and Gardner 2008; Uy et al. 2011; Tam et al. 2014; Yang and Ma 2013). The regression analysis is summarized in Fig. 4.15, in which the test data of the present study is also included and the dashed line represents the Eq. (4.1b). It can be seen that the current data are in the same scatter band as the existing data.


Fig. 4.15. Regression analysis of existing data for concrete-filled SS tubes

A comparison between the experimental capacity and predicted capacity by the method proposed in Chapter 3 is presented in Table 4.8. The data in Chapter 3 is also included in Table 4.8. As shown in Table 4.8, this method slightly over estimate the capacity of fully SWSSC-filled SS tubes, especially for the specimens in the present study. One reason is that the load corresponding to 5% axial strain is adopted as N_t but the applied load can still increase slightly after the axial strain of 5% for some fully SWSSC-filled SS tubes (see Fig. 4.8(a)). In general, this method can provide reasonable estimation of the ultimate capacity of fully SWSSC-filled SS tubes.

Data source	Specimen	<i>f</i> _{0.2} (MPa)	f _c ' (MPa)	ξ	$f_{\rm scy}$ (MPa)	$N_{\rm t}$ (kN)	$N_{\rm p}({\rm kN})$	$N_{\rm p}/N_{\rm t}$
	S50-C	228.2	35.8	1.87	134.6	235	274	1.16
This abortor	S101-C	225.7	35.8	0.75	78.4	570	640	1.12
This chapter	S114-C	280.7	35.8	0.83	82.3	766	841	1.10
	S165-C	281.1	35.8	0.64	72.7	1449	1620	1.12
	S101-C	324.4	31.4	1.26	91.2	729	734	1.01
Chapter 3	S114-C	270.3	31.4	0.94	77.2	800	786	0.98
	S165-C	280.1	31.4	0.71	66.9	1522	1487	0.98
Mean								1.07
COV								0.070

Table 4.8. Comparison between experimental capacity and estimated capacity of fully SWSSC-filled SS tube

4.5.2 SWSSC-filled double-skin tubes (SS as the outer tube)

The formulae presented in Chapter 3 is adopted herein to estimate the ultimate capacity of SWSSCfilled double-skin tubes (SS as the outer tube). It is assumed in Chapter 3 that the capacity of doubleskin tubes is the summation of the capacity of outer SS tube with sandwiched concrete (with appropriate modification) and the capacity of inner SS or FRP tube. The detailed formulae can be found in Chapter 3. They are summarised here for the convenience of readers.

$$N_{\rm p} = (A_{\rm o} + A_{\rm c})f_{\rm scy} + A_{\rm i}f_{\rm yi}$$
(4.2a)

$$f_{\rm scy} = \frac{\alpha}{1+\alpha} \chi^2 f_{\rm yo} + \frac{1+\alpha_{\rm n}}{1+\alpha} (1.14+1.4\xi) f_{\rm c}'$$
(4.2b)

$$\xi = \frac{A_{\rm o}f_{\rm yo}}{A_{\rm cn}f_{\rm c}'} \tag{4.2c}$$

$$\alpha = A_{\rm o} / A_{\rm c} \tag{4.2d}$$

$$\alpha_{\rm n} = A_{\rm o} / A_{\rm cn} \tag{4.2e}$$

where A_0 is the outer tube cross-section area, A_c is the cross-section area of concrete, A_i is the inner tube cross-section area, A_{cn} is the nominal concrete area $(=\pi D_0^2/4)$, f_{yo} is strength of outer tube $(=f_{0.2}$ for SS), f_{yi} is strength of inner tube, f_c ' is unconfined concrete strength, ξ is confinement factor, χ is void ratio $(=D_i/D_o)$, and f_{scy} is nominal yield strength of composite sections. Due to the considerable strain hardening behaviour of SS material, the strength of the inner SS tube (f_{yi}) is approximately taken as the average value of yield strength $(f_{0.2})$ and ultimate strength (f_u) . The strength of the inner FRP tube (f_{yi}) is taken as the averaged residual strength (σ_{res}) obtained from compression test on hollow section tubes.

Data source	Specimen	f _{yo} (MPa)	f _{yi} (MPa)	fc' (MPa)	ξ	χ	f _{scy} (MPa)	N _t (kN)	N _p (kN)	$N_{\rm p}/N_{\rm t}$
	S114-S50-C	280.7	395.1	39.4	0.75	0.45	90.8	852	925	1.09
	S165-S101-C	281.1	441.0	39.4	0.58	0.61	85.4	1314	1589	1.21
This shouten	S114-C50-C	280.7	88.5	39.4	0.75	0.44	90.7	735	782	1.06
This chapter	S165-C101-C	281.1	88.5	39.4	0.58	0.59	84.9	1271	1301	1.02
	S114-B50-C	280.7	59.3	32.8	0.90	0.44	83.2	676	711	1.05
	S165-B101-C	281.1	59.3	32.8	0.70	0.59	77.7	1145	1173	1.02
	S114-S50-C	270.3	462.5	31.4	0.93	0.42	80.6	909	864	0.95
Charter 2	S165-S101-C	280.1	485.8	31.4	0.72	0.60	76.1	1409	1491	1.06
Chapter 5	S114-G50-C	270.3	113.6	31.4	0.96	0.45	82.7	799	731	0.91
	S165-G101-C	280.1	53.9	31.4	0.72	0.60	76.0	1167	1143	0.98
Mean										1.04
COV										0.079

Table 4.9. Comparison between experimental capacity and estimated capacity of SWSSC-filled double-skin tubes (SS as the outer tube)

The comparison between the test capacity (N_t) and predicted capacity (N_p) is summarized in Table 4.9, in which both the data in the present study and in Chapter 3 are included. The averaged value of N_p/N_t is 1.04 with coefficient of variance (COV) of 0.079, which suggests the design method in Chapter 3 can be adopted as a unified approach to predict the ultimate capacity of SWSSC-filled double-skin tubes with SS as the outer tube no matter what kind of inner tube (SS, GFRP, BFRP or CFRP) it is.

4.5.3 Fully SWSSC-filled CFRP and BFRP tubes

As the longitudinal strength of the FRP tube cannot be ignored, the current design method for concrete confined by FRP wraps, in which the longitudinal strength is ignored, is not suitable for fully concrete-filled FRP tubes. Based on the available data, Chapter 3 proposed a new method to estimate the ultimate capacity of fully SWSSC-filled GFRP tubes. However, no design method has been reported for concrete-filled CFRP or BFRP tubes. Due to the limitation of available data, new method is not proposed in the present study. The method proposed by Chapter 3 is adopted to estimate the ultimate capacity of fully SWSSC-filled CFRP and BFRP tubes. The formulae are listed below:

$$N_{\rm p} = f_{\rm un} (A_{\rm c} + A_{\rm o} \frac{f_{\rm ul}}{f_{\rm uh}})$$
(4.3a)

$$\frac{f_{\rm un}}{f_{\rm c}'} = 1.12 + 2.64 \frac{f_l}{f_{\rm c}'} \tag{4.3b}$$

$$f_l = \frac{2f_{\rm uh}t_{\rm o}}{D_{\rm o}} \tag{4.3c}$$

where N_p is predicted capacity, A_c is cross-section area of concrete, A_o is cross-section area of outer tube (i.e. FRP tube), f_{ul} is ultimate strength in longitudinal direction, f_{uh} is ultimate strength in hoop direction, f_c ' is unconfined concrete strength, f_l is confining pressure, D_o is outer diameter of the outer tube, t_o is thickness of the outer tube, and f_{un} is nominal yielding strength. The relationship between f_{un}/f_c ' and f_l/f_c ' (i.e. Eq. (4.3b)) was determined by regression analysis as shown in Fig. 4.16, in which the dashed line represents Eq. (4.3b) and the data (Chapter 3; Fam and Rizkalla 2001; Zhang et al. 2015) for CFRP and BFRP tubes is also presented. As shown in Fig. 4.16, Eq. (4.3b) underestimates the confinement effect provided by CFRP tubes in a few cases.

A comparison between the test capacity and predicted capacity for fully SWSSC-filled FRP tubes is summarized in Table 4.10, in which the test data for GFRP is also included. Table 4.10 indicates that this method underestimates the capacity of fully SWSSC-filled CFRP tubes and the prediction for fully SWSSC-filled BFRP tubes is reasonable. This conservative prediction may be caused by the different material properties (e.g. f_{uh}) of CFRP and GFRP. Therefore, more study is needed on concrete-filled CFRP tubes to propose more appropriate design methods.



Fig. 4.16. Regression analysis of existing data for concrete-filled FRP tubes

lubes									
Data source	Specimen	f _{ul} (MPa)	f _{uh} (MPa)	fc' (MPa)	fi (MPa)	f _{un} (MPa)	N _t (kN)	N _p (kN)	$N_{\rm p}/N_{\rm t}$
	С50-С	242.9	592.8	35.8	66.0	214.3	388	375	0.97
	C101-C	242.9	592.8	35.8	33.3	128.1	1131	940	0.83
	C114-C	242.9	592.8	35.8	28.5	115.2	1416	1122	0.79
This shoutes	C165-C	242.9	592.8	35.8	20.9	95.3	2372	1794	0.76
This chapter	B50-C	124.0	331.1	32.8	35.8	131.2	259	225	0.87
	B101-C	124.0	331.1	32.8	19.3	87.7	656	641	0.98
	B114-C	124.0	331.1	32.8	16.1	79.1	825	767	0.93
	B165-C	124.0	331.1	32.8	11.4	66.7	1345	1248	0.93
	G50-C	217.6	308.8	31.4	37.1	133.2	244	255	1.05
Charten 2	G101-C	217.6	308.8	31.4	19.3	86.1	670	653	0.98
Chapter 5	G114-C	217.6	308.8	31.4	16.8	79.4	813	802	0.99
	G165-C	217.6	308.8	31.4	12.3	67.5	1336	1296	0.97
Mean									0.92
COV									0.097

Table 4.10. Comparison between experimental capacity and estimated capacity of fully SWSSC-filled FRP tubes

4.5.4 SWSSC-filled double-skin tubes (CFRP or BFRP as the outer tube)

Past research (Fam and Rizkalla 2001; Li et al. 2016) on concrete-filled double-skin tubes (GFRP as the outer tube) indicated that it is reasonable to assume that the ultimate capacity of the double-skin tubes is equal to the summation of the capacity of outer tube with sandwiched concrete, which behaves like fully concrete-filled tubes, and the capacity of inner tubes. The method proposed in Chapter 3 is adopted to estimate the ultimate capacity of double-skin tubes in the present study. They are summarised here for the convenience of readers.

$$N_{\rm p} = N_{\rm co} + N_{\rm i} \tag{4.4a}$$

$$N_{\rm i} = f_{\rm yi} A_{\rm i} \tag{4.4b}$$

where N_{co} is the capacity of the outer tube with sandwiched concrete, which can be determined by Eq. (4.3a), A_i is the cross-section area of the inner tube, and N_i is the capacity of the inner tube.

A comparison between the experimental capacity (N_t) and prediction (N_p) is summarized in Table 4.11, in which f_{yi} is strength of inner tube, f_{ul} is longitudinal strength of the outer tube, f_{uh} is hoop strength of the outer tube, f_c ' is unconfined concrete strength, f_l is confining pressure, χ is void ratio (D_i/D_o). If the inner tube is SS, f_{yi} is the averaged value of yield strength ($f_{0.2}$) and ultimate strength (f_u). For the inner FRP tube, f_{yi} is taken as the averaged residual strength (σ_{res}).

tubes (I KI as	the outer tube)									
Data source	Specimen	f _{yi} (MPa)	f _{ul} (MPa)	f _{uh} (MPa)	fc' (MPa)	fı (MPa)	χ	Nt (kN)	N _p (kN)	$N_{\rm p}/N_{\rm t}$
	C114-S50-C	395.1	242.9	592.8	39.4	28.5	0.44	1375	1101	0.80
	C165-S101-C	441.0	242.9	592.8	39.4	20.9	0.64	1698	1442	0.85
	С114-С50-С	165.7	242.9	592.8	39.4	28.5	0.44	1175	992	0.84
This shouten	C165-C101-C ^a	165.7	242.9	592.8	39.4	20.9	0.63	1219	1233	1.01
This chapter	B114-S50-C	395.1	124.0	331.1	32.8	16.1	0.44	884	788	0.89
	B165-S101-C	441.0	124.0	331.1	32.8	11.4	0.65	1053	1086	1.03
	B114-B50-C ^a	91.7	124.0	331.1	32.8	16.1	0.44	651	648	1.00
	B165-B101-C ^a	91.7	124.0	331.1	32.8	11.4	0.63	703	805	1.14
	G114-S50-C	462.5	217.6	308.8	31.4	15.6	0.42	872	815	0.93
Chantan 2	G165-S101-C	485.8	217.6	308.8	31.4	11.4	0.64	1301	1160	0.89
Chapter 3	G114-G50-C	164.6	217.6	308.8	31.4	15.7	0.45	795	688	0.87
	G165-G101-C	135.8	217.6	308.8	31.4	12.2	0.63	880	893	1.01
Mean										0.94
COV										0.106

Table 4.11. Comparison between experimental capacity and estimated capacity of SWSSC-filled double-skin tubes (FRP as the outer tube)

^a: No tube rupture.

As shown in Table 4.11, the prediction by the method in Chapter 3 is reasonable (Mean=0.94, COV=0.106), although the prediction for double-skin tubes with CFRP as the outer tube is slightly conservative.

As mentioned before some double-skin tubes (i.e. C165-C101-C, B114-B50-C, and B165-B101-C) did not fail by tube rupture. It should pointed out that the method adopted herein is based on the assumption that the outer FRP tube can reach the hoop strength (f_{uh}). Although the prediction is quite close the experimental data, more studies are needed to investigate the hoop stress in the outer tube for concrete-filled double skin tubes with FRP as both the outer and inner tubes.

4.6 Conclusions

This chapter presents an experimental investigation on SWSSC-filled SS, CFRP and BFRP tubular stub columns under axial compression. Both the fully SWSSC-filled tubes and SWSSC-filled double-

skin tubes with different combinations of tube materials were tested. Several conclusions can be made as follows:

(1) The strength and ductility of SWSSC-filled tubes are significantly enhanced in comparison with hollow section tubes and plain concrete.

(2) Among the three FRPs investigated, CFRP has the highest ultimate strength and elastic moduli in both longitudinal and transverse directions. BFRP has the lowest ultimate strength and elastic modulus in longitudinal direction, while the material properties of BFRP in hoop direction are similar to those of GFRP. The ultimate strain of CFRP is much less than that of BFRP and GFRP.

(3) The confinement effect provided by SS tubes is lower than that by CFRP and BFRP tubes, but the confinement can be maintained for larger axial strain for SS tubes. The strength enhancement and ductility of SWSSC-filled CFRP tubes are higher than those of BFRP tubes mainly due to its higher hoop strength. The confinement provided by BFRP and GFRP tubes is quite similar.

(4) For some of the double-skin tubes with FRP as both the outer and inner tubes, the outer tube rupture did not occur as the inner FRP cannot effectively restrain the lateral expansion of concrete after the axial strain reaches around 0.03.

(5) As the diameter-to-thickness ratio of tubes increases, the level of confinement reduces for all the four types of tubes (SS, GFRP, BFRP and CFRP). When compared with fully filled tubes, the confinement provided by the double skin tubes start to decrease at large deformation due to the buckling of inner FRP tubes. The influence of inner tube on confinement is not significant unless they are slender FRP tubes after an axial strain of 0.03.

(6) The formulae proposed in Chapter 3 can be adopted as a unified approach to estimate the ultimate capacity of SWSSC-filled SS, GFRP, CFRP and BFRP tubes. In general, the prediction is in reasonable agreement to the test data although the capacity estimation of SWSSC-filled CFRP tubes is slightly conservative.

Research is being conducted on the long-term behavior of SWSSC and the shrinkage effect on the confinement. The existing work in the literature (Han et al. 2014; Han et al. 2012; Hou et al. 213; Li et al. 2015) on the behaviour of normal composite concrete-filled steel tubular columns under chloride corrosion will be consulted.

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References

ACI 440.2R-08 (2008), Guide for the design and construction of externally bonded FRP systems for strengthening concrete structures, American Concrete Institute, Farmington Hills, MI.

AS 1391 (2007), Metallic materials-tensile testing at ambient temperature, Standards Australia, Sydney.

AS/NZS 4673 (2001), Cold-formed stainless steel structures, Standards Australia, Sydney.

ASTM D2290-12 (2012), Standard test method for apparent hoop tensile strength of plastic or reinforced plastic pipe, American Society for Testing and Materials, West Conshohocken, PA.

ASTM D3039/D3039M-14 (2014), Standard test method for tensile properties of polymer matrix composite materials, American Society for Testing and Materials, West Conshohocken, PA.

Chen, Z.F., Wan, L.L., Lee, S., Ng, M., Tang, J.M., Li, M. and Lee, L. (2008), Evaluation of CFRP, GFRP and BFRP material systems for the strengthening of RC Slabs, Journal of Reinforced Plastics and Composites, 27(12), 1233-1243.

Fam, A.Z. and Rizkalla, S.H. (2001), Behavior of axial loaded concrete-filled circular fiber-reinforced polymer tubes, ACI Strucural Journal, 98(3), 280-289.

Han, L.H., Yao, G.H. and Zhao, X.L. (2005), Tests and calculations for hollow structural steel (HSS) stub columns filled with self-consolidating concrete (SCC), Journal of Constructional Steel Research, 61, 1241-1269.

Han, L.H., Hou, C. and Wang, Q.L. (2012), Square concrete filled steel tubular (CFST) members under loading and chloride corrosion: Experiments, Journal of Constructional Steel Research, 71, 11-25.

Han, L.H., Hou, C.C. and Wang, Q.L. (2014), Behavior of circular CFST stub columns under sustained load and chloridecorrosion, Journal of Constructional Steel Research, 103, 23-36.

Hou, C., Han, L.H. and Zhao, X.L. (2013), Full-range analysis on square CFST stub columns and beams under loading and chloride corrosion, Thin-Walled Structures, 68, 50-64.

Kaushik, S.K. and Islam, S. (1995), Suitability of sea water for mixing structural concrete exposed to a marine environment, Cement and Concrete Composites, 17(3), 177-185.

Kupwade-Patil K. and Allouche, E. (2013), Impact of alkali silica reaction on fly ash-based Geopolymer concrete, Journal of Materials in Civil Engineering, 25(1), 131-139.

Lam, D. and Gardner, L. (2008), Structural design of stainless steel concrete filled columns, Journal of Constructional Steel Research, 64(11), 1275-1282.

Li, W., Han, L.H. and Zhao, X.L. (2015), Behavior of CFDST stub columns under preload, sustained load and chloride corrosion, Journal of Constructional Steel Research, 107, 12-23.

Li, Y.L., Zhao, X.L., Raman Singh, R.K. and Al-Saadi, S. (2016), Experimental study on seawater and sea sand concrete filled GFRP and stainless steel tubular stub columns, Thin-Walled Structures 106, 390-406.

Lopresto, V., Leone, C. and De Iorio, I. (2011), Mechanical characterisation of basalt fibre reinforced plastic, Composites: Part B, 42, 717-723.

Mohammed, T.U.T., Hamada, H. and Yamaji, T. (2004), Performance of seawater-mixed concrete in the tidal environment, Cement and Concrete Research, 34(4), 593-601.

Ozbakkaloglu, T., Lim, J.C. and Vincent, T. (2013), FRP-confined concrete in circular sections: Review and assessment of stress–strain models, Engineering Structures, 49, 1068-1088.

Ozbakkaloglu, T. and Xie, T. (2016), Geopolymer concrete-filled FRP tubes: Behavior of circular and square columns under axial compression, Composites Part B, 96, 215-230.

Tam, V.W.Y., Wang, Z. and Tao, Z. (2014), Behaviour of recycled aggregate concrete filled stainless steel stub columns, Materials and Structures, 47(1-2), 293-310.

Teng, J.G. (2014), Performance enhancement of structures through the use of fibre-reinforced polymer (FRP) composites, in: Proceedings of 23rd Australasian Conference on the Mechanics of Structures and Materials (ACMSM23), Lismore, Australia, keynote presentation.

Teng, J.G., Jang, T., Lam, L. and Luo, Y.Z. (2009), Refinement of a design-oriented stress–strain model for FRP-confined concrete, Journal of Composites for Construction, 13(4), 269-278.

Teng, J.G. and Lam, L. (2004), Behavior and modeling of fiber reinforced polymer-confined concrete, Journal of Structural Engineering, 130(11), 1713-1723.

Teng, J.G., Yu, T., Dai, J.G. and Chen, G.M. (2011), FRP composites in new construction: current status and opportunities, in: Proceedings of 7th National Conference on FRP Composites in Infrastructure (Supplementary Issue of Industrial Construction), keynote presentation.

Teng, J.G., Yu, T., Wong, Y.L. and Dong, S.L. (2007), Hybrid FRP–concrete–steel tubular columns: Concept and behaviour, Construction and Building Materials, 21(4), 846-854.

Uy, B., Tao, Z. and Han, L.H. (2011), Behaviour of short and slender concrete-filled stainless steel tubular columns, Journal of Constructional Steel Research, 67(3), 360-378.

Yang, Y.F. and Ma, G.L. (2013), Experimental behaviour of recycled aggregate concrete filled stainless steel tube stub columns and beams, Thin-Walled Structures, 66, 62-75.

Zhang, B., Yu, T. and Teng, J.G. (2015), Behavior of concrete-filled FRP tubes under cyclic axial compression, Journal of Composites for Construction, 2015, 19(3), 04014060.

Zhao, X.L. and Han, L.H. (2006), Double skin composite construction, Progress in Structural Engineering and Materials, 8(3), 93-102.

Zhao, X.L., Han, L.H. and Lu, H. (2010), Concrete-filled tubular members and connections, CRC Press, London.

Chapter 5

Axial compression tests on seawater and sea sand concretefilled double-skin stainless steel circular tubes

Abstract

Seawater and sea sand concrete (SWSSC) filled double-skin stainless steel (SS) tubes consist of two concentric circular SS tubes, sandwiching SWSSC between them. This chapter presents an experimental study on SWSSC-filled double-skin SS concrete-filled steel tubes (CFDSTs) under axial compression. Unfilled circular hollow sectional (CHS) specimens, fully-filled tubes (CFSTs) and double-skin tubes without inner tube (CFHT) were also tested for comparison purpose. Load-axial strain curves, stress in concrete, post-peak behaviour and energy absorption are investigated in this study. The effects of some key parameters, including confinement factor, void ratio and tube slenderness ratio, on the structural behaviour of SWSSC-filled SS tubes are discussed. It is found that the ultimate stress in concrete is mainly affected by confinement factor and the post-peak behaviour depends on both confinement factor and inner tube slenderness. Formulas are proposed to estimate the load carrying capacity of SWSSC-filled SS circular tubes in compression.

Keywords

Seawater and sea sand concrete (SWSSC); stainless steel (SS); double-skin tubes; axial compression

Nomenclature

$A_{ m c}$	=	Concrete area
$A_{\rm cn}$	=	Nominal concrete area
$A_{ m i}$	=	Inner tube cross-sectional area
$A_{ m o}$	=	Outer tube cross-sectional area
$D_{ m i}$	=	Inner tube diameter
$D_{ m o}$	=	Outer tube diameter
$E_{0.2}$	=	Tangent modulus at f_y
$E_{ m o}$	=	Initial elastic modulus
$f_{\rm c}$ '	=	Unconfined concrete strength
f_{cc} '	=	Confined concrete strength
$f_{\rm ck}$	=	Characteristic concrete strength
$f_{ m scy}$	=	Nominal yielding strength of composite section
$f_{ m u}$	=	Ultimate stress
f_{y}	=	Yielding stress
$f_{ m yi}$	=	Yielding stress of inner tube
$f_{ m yo}$	=	Yielding stress of outer tube
L	=	Specimen length
т	=	Constant
n	=	Constant
$N_{\rm AS}$	=	Capacity of CHS determined by AS/NZS 4673:2001
$N_{\rm ASCE}$	=	Capacity of CHS determined by SEI/ASCE8-02
$N_{\rm CSM}$	=	Capacity of CHS determined by Continuous strength method (CSM)
$N_{\rm EN}$	=	Capacity of CHS determined by EN1993-1-4:2006

$N_{ m i}$	=	Predicted capacity of inner SS tube in CFDST
$N_{ m min}$	=	Minimum load after reaching N_t and before reaching 5% axial strain
$N_{ m p}$	=	Predicted capacity of SWSSC-filled tubes
$N_{ m t}$	=	Experimental ultimate capacity
$N_{ m y}$	=	Yielding capacity of CHS
ti	=	Inner tube thickness
to	=	Outer tube thickness
З	=	Strain
ε_0	=	Elastic strain at f_y
$\varepsilon_{ m LB}$	=	Ultimate strain of CHS determined by CSM
\mathcal{E}_{u}	=	Ultimate strain
ε_{y}	=	Yielding strain
$\lambda_{\rm c}$	=	Cross-section slenderness ratio for CHS
λ_{i}	=	Inner tube slenderness ratio
λ_{o}	=	Outer tube slenderness ratio
ξ	=	Confinement factor
σ	=	Stress
υ	=	Poisson's ratio
χ	=	Void ratio

5.1 Introduction

Concrete-filled steel tubes (CFSTs), which consist of outer steel tube and in-filled concrete, have been widely applied in civil engineering, such as bridge piers and high-rise building columns (Zhao et al. 2010). Since the concrete is confined by steel tube and buckling of steel tube is delayed (or prevented) by the in-filled concrete, CFSTs have the advantages of high strength and high ductility in comparison to individual concrete or steel tube. Different shapes (e.g., circular, square, and rectangular) of steel tubes have been used in CFSTs and the circular tubes have been found to provide the largest confinement effect. In recent decades, the concept of concrete filled double-skin steel tube (CFDST), which consists of two concentric steel tubes with concrete filled between them, was developed (Zhao and Han 2006). The voids in CFDST can reduce the self-weigh and enhance the bending stiffness, which is of great significance in long columns.

The consumptions of fresh water and river sand as raw materials for concrete and the emission of CO₂ in producing cement have exacerbated the resource shortage and caused environmental impacts (e.g., damage on river ecosystem). The application of seawater and sea sand concrete (SWSSC) with geopolymer, which contains seawater, sea sand, alkali-activated geopolymer, has attracted great attention of researchers (Li et al. 2016a, b, 2018; Xiao et al. 2017; Wang et al. 2017a, b) in order to mitigate the environmental problems. The initial cost for stainless steel is higher than carbon steel. However, if considering the environmental benefits of using seawater and sea sand concrete and the

maintenance cost caused by corrosion of carbon steel, the life-cycle cost of SWSSC-filled SS tubes is still economically feasible. However, SWSSC in CFSTs may pose a serious durability issue since the chloride in SWSSC can vastly impair the corrosion resistance of the carbon steel. One option is to replace the carbon steel tubes by FRP tubes (Teng et al. 2011; Teng 2014; Chen et al. 2017; Li et al. 2018b) and the other solution is to use stainless steel (SS) tubes. SWSSC-filled double-skin stainless steel tubes can also be potentially applied in marine environments, such as coastal infrastructures and ocean platforms. Past studies (Li et al. 2018a; Nishida et al. 2015) indicate that the short-term mechanical properties of SWSSC are similar to those of ordinary Portland cement concrete.

Extensive studies have been conducted on concrete-filled carbon steel tubes and they are well reviewed in (Zhao et al. 2010; Shams and Saadeghvaziri 1997). It is generally agreed that the strength of concrete core is enhanced due to confinement effect and the buckling of steel tube is postponed or eliminated due to the in-filled concrete. The design method for CFSTs has been specified in standards, such as AS5100 (2017), ANSI/AISC360 (2010) and EC4 (2004). Recently, several studies (experimental: Tao et al. 2004; Wei et al. 1995a, b; Yang et al. 2012; Zhao et al. 2002; Zhao et al. 2010; Uenaka et al. 2010; numerical: Han et al. 2010; Huang et al. 2010) have been conducted for CFDST stub columns (circular carbon steel tubes as both inner and outer tubes) under axial compression and the effects of tube slenderness, void ratio, confinement, load position were investigated. Design formulas were also proposed to estimate the strength of CFDSTs, among which simple superposition method was adopted by Zhao et al. (2002) and composite action was accounted for by Tao et al. (2004) and Wei et al. (1995). The performance of concrete-filled circular stainless steel tubular stub columns under axial compression (Lam and Gardner 2008; Uy et al. 2011; Tam et al. 2014; Han et al. 2011) was found to be quite good and current design codes to be conservative for concrete-filled SS tubes. There is no published work on the short term performance of concrete-filled double skin stainless steel tubes apart from 4 preliminary tests mentioned in Chapters 3 and 4, although some studies have been conducted on concrete fully filled stainless steel (Lam and Gardener 2008; Uy et al. 2011; Tam et al. 2014) and double skin tubes with SS as the outer tube and carbon steel as the inner tube (Han et al. 2011). This situation is confirmed by a current review paper on concrete-filled stainless steel tubes (Han et al. 2018). Due to the influence of the voids in double-skin tubes, the theory for fully filled tubes cannot be directly used for double-skin tubes. Furthermore, the material property (i.e., stress-strain relationship) of stainless steel is different from that of carbon steel. Stainless steel has a rounded shape of stress-strain curve without obvious yielding point. The design theory for concrete-filled double skin carbon steel tubes cannot be directly applied for double skin SS

tubes. Therefore there is a need to study the short-term behaviour of SWSSC filled double skin SS tubes.

As aforementioned, in using SWSSC for CFDST, conventional carbon steel tubes are not suitable due to their low corrosion resistance. Stainless steel is more suitable because of its superior corrosion resistance. As the stress-strain curve of SS is different from that for carbon steel, the structural behaviour of concrete-filled double-skin SS tubes is different from that of carbon steel CFDSTs. A comprehensive study is needed for CFDST (circular SS tubes as both inner and outer tubes) and proper design method is required to estimate its load-carrying capacity under axial compression. Present study focuses on the structural behaviour of seawater and sea sand concrete-filled double-skin circular stainless steel tubular stub columns ("CFDST" in short in the following) under axial compression. Ten fully filled specimens (CFSTs) and five double-skin specimens without inner SS tube (CFHTs) were also tested for comparison purpose. The effects of parameters, such as confinement, void ratio, tube slenderness, on the structural behaviour (e.g., load-strain curves, ultimate capacity, concrete strength, energy absorption) were investigated. Finally, design formulas were proposed for prediction of the ultimate capacity of both fully-filled and double-skin tubes.

5.2 Experimental programme

5.2.1 Materials

The stainless steel tubes were made of AISI316 grade austenitic stainless steel with nominal yield strength (0.2% proof strength, f_y) of 205 MPa in according with AS/NZS 4673:2001 (2001). Tensile coupon test was conducted on each size of SS tubes to obtain their material properties and the test results will be discussed in Section 5.3.1.

Alkali-activated, slag based seawater and sea sand concrete (SWSSC) was adopted in the present study. The concrete mixture is: slag (360 kg/m³), seawater (190 kg/m³), sea sand (830 kg/m³), coarse aggregate with maximum size of 14 mm (1130 kg/m³), sodium meta-silicate (38.4 kg/m³) and hydrated lime slurry (14.4 kg/m³). More details of the mixture can be found in Chapter 3 with regards to the particle size distribution of sea sand, chemical compositions of the raw materials and concrete mixing procedures. A total of four batches of SWSSC were casted and the average 28-day strength (f_c ') for each batch, which was determined from compression test on three identical concrete cylinders (diameter of 100 mm and height of 200 mm) according to AS 1012.9:2014 (2014), was 43.1 MPa, 41.8 MPa, 41.4 MPa and 41.6 MPa respectively. For simplicity, an averaged 28-day concrete strength of 42.0 MPa was adopted in this study since the strength variations between the batches were very small (with standard deviation of 0.7 MPa).

5.2.2 Specimens

Both the unfilled circular hollow sectional (CHS) specimens and SWSSC-filled SS tubes, including fully-filled tubes (CFSTs), double-skin tubes (CFDSTs) and double-skin tubes without inner SS tube (CFHTs) were tested in the present study. The cross-section configurations of the specimens are illustrated in Fig. 5.1. In order to eliminate the influence of global buckling, the specimen length (*L*) was about three times its outer diameter (D_0) except SWSSC-filled specimens with outer tube size of 203×2, whose length was 400 mm. The purpose for setting a lower value of L/D_0 (2 to 3) for these stub column specimens is to eliminate the influence of any overall buckling or boundary end effect and to reduce the self-weight for easy concrete compaction and specimen carriage.



Table 5.1. Specimen details for unfilled CHS stub columns

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Specimen	$D_{\rm o}$ (mm)	$t_{\rm o} ({\rm mm})$	L (mm)	$N_{\rm t}({\rm kN})$
H50×1.6	49.6	1.53	150	91
H50×3	50.9	3.07	150	202
H76×1.6	76.2	1.66	230	144
H89×3	89.2	3.22	270	274
H101×1.6	101.8	1.70	300	184
H101×3	101.9	2.79	300	254
H114×3	114.1	2.79	350	335
H152×1.6	152.6	1.60	450	247
H168×3	168.4	3.22	450	537
H203×2	202.7	1.99	600	347

	Outer t	ube			Inner to	ube					L Nt	f.c.'	
Specimen	Do (mm)	t _o (mm)	f _{yo} (MPa)	λο	Di (mm)	t _i (mm)	f _{yi} (MPa)	λ_i	χ	ξ	(mm)	(kN)	(MPa)
50×1.6-F	49.6	1.53	376.5	49					1	1.22	150	209	69.7
50×3-F	50.9	3.07	228.9	15					1	1.60	150	244 ^a	57.6
76×1.6-F	76.2	1.66	398.9	73					1	0.88	230	388	58.4
89×3-F	89.2	3.22	259.2	29					1	1.00	270	548 ^a	58.9
101×1.6-F	101.8	1.70	353.3	85					1	0.59	300	617	56.9
101×3-F	101.9	2.79	226.0	33					1	0.64	300	610	54.6
114×3-F	114.1	2.79	281.2	46					1	0.71	350	831	56.3
152×1.6-F	152.6	1.60	314.5	120					1	0.32	450	1050	45.8
168×3-F	168.4	3.22	281.5	59					1	0.54	450	1635	55.9
203×2-F	202.7	1.99	304.0	124					1	0.29	600	1787	46.4
101×1.6-50×1.6-D	101.8	1.70	353.3	85	49.6	1.53	376.5	49	0.50	0.59	300	583	55.4
101×1.6-50×3-D	101.8	1.70	353.3	85	50.9	3.07	228.9	15	0.52	0.59	300	634	59.9
101×3-50×1.6-D	101.9	2.79	226.0	33	49.6	1.53	376.5	49	0.52	0.64	300	593	55.9
101×3-50×3-D	101.9	2.79	226.0	33	50.9	3.07	228.9	15	0.53	0.64	300	681 ^a	58.9
114×3-50×1.6-D	114.1	2.79	281.2	46	49.6	1.53	376.5	49	0.46	0.71	350	804	55.6
114×3-50×3-D	114.1	2.79	281.2	46	50.9	3.07	228.9	15	0.47	0.71	350	902 ^a	58.3
152×1.6-50×1.6-D	152.6	1.60	314.5	120	49.6	1.53	376.5	49	0.33	0.32	450	1055	46.4
152×1.6-76×1.6-D	152.6	1.60	314.5	120	76.2	1.66	398.9	73	0.51	0.32	450	997	47.2
152×1.6-101×1.6-D	152.6	1.60	314.5	120	101.8	1.70	353.3	85	0.68	0.32	450	825	42.3
152×1.6-101×3-D	152.6	1.60	314.5	120	101.9	2.79	226.0	33	0.68	0.32	450	882	47.0
168×3-50×1.6-D	168.4	3.22	281.5	59	49.6	1.53	376.5	49	0.31	0.54	450	1569	52.7
168×3-76×1.6-D	168.4	3.22	281.5	59	76.2	1.66	398.9	73	0.47	0.54	450	1470	51.8
168×3-89×3-D	168.4	3.22	281.5	59	89.2	3.22	259.2	29	0.55	0.54	450	1464	53.0
168×3-101×1.6-D	168.4	3.22	281.5	59	101.8	1.70	353.3	85	0.63	0.54	405	1332	54.0
168×3-101×3-D	168.4	3.22	281.5	59	101.9	2.79	226.0	33	0.63	0.54	450	1354	53.8
168×3-114×3-D	168.4	3.22	281.5	59	114.1	2.79	281.2	46	0.70	0.54	450	1319	50.9
203×2-50×3-D	202.7	1.99	304.0	124	50.9	3.07	228.9	15	0.26	0.29	400	1653	40.9
203×2-76×1.6-D	202.7	1.99	304.0	124	76.2	1.66	398.9	73	0.38	0.29	400	1658	42.2
203×2-101×3-D	202.7	1.99	304.0	124	101.9	2.79	226.0	33	0.51	0.29	400	1625	46.9
203×2-152×1.6-D	202.7	1.99	304.0	124	152.6	1.60	314.5	120	0.77	0.29	400	1142	43.1
101×1.6-50-Н	101.8	1.70	353.3	85	59.3				0.60	0.59	300	410	46.3
101×3-50-Н	101.9	2.79	226.0	33	60.0				0.62	0.64	300	415	48.4
152×1.6-76-Н	152.6	1.60	314.5	120	82.8				0.55	0.32	450	763	42.9
203×2-101-Н	202.7	1.99	304.0	124	108.3				0.54	0.29	400	1159	37.2
203×2-152-Н	202.7	1.99	304.0	124	155.8				0.78	0.29	400	626	23.3

Table 5.2. Specimen details for SWSSC-filled tubes

^a: The load corresponding to 5% axial strain was adopted as $N_{\rm t}$

The SS tubes were first cut into required length using a diamond saw and these CHS specimens were then directly tested. For SWSSC-filled specimens, SS tubes were mounted on a timber plate and then cast with SWSSC using a vibration table to compact concrete. Concrete cylinders were also cast at the same time. Timber plates were removed one day after the casting and the specimens with plastic films on top were stored at room temperature until compression test. The central holes in CFHT specimens were achieved by using cardboard tubes, which were removed after concrete hardening. The dimensions of CHS specimens and SWSSC-filled specimens are listed in Tables 5.1 and 5.2 respectively, together with the ultimate capacity. In Table 5.2, the slenderness ratio of outer (λ_0) and inner SS tubes (λ_i), confinement factor (ζ) and void ratio (χ) are also presented and their definitions are:

$$\lambda_o = \frac{D_o}{t_o} \cdot \frac{f_{yo}}{250} \tag{5.1}$$

$$\lambda_i = \frac{D_i}{t_i} \cdot \frac{f_{yi}}{250} \tag{5.2}$$

$$\xi = \frac{f_{yo}A_o}{f_c'A_{cn}} \tag{5.3}$$

$$\chi = \frac{D_i}{D_o - 2t_o} \tag{5.4}$$

The definition of slenderness ratio (Eqs. (5.1-2)) is adopted from AS4100 (2016) for carbon steel. A_{cn} in Eq. (5.3) is nominal concrete area (= π (D_o -2 t_o)²/4). It is necessary to emphasize that in Eq. (5.3) experimental cylindrical strength of concrete (f_c ') is used, which is different from the characteristic concrete strength (f_{ck}) used in Uy et al. (2011) and Han et al. (2011), in which f_{ck} was took as 67% of the compressive strength of cubic blocks.

The label of CHS specimens is "H" followed by nominal SS tube size. SWSSC-filled specimens are labelled in the form of "nominal outer SS tube size – nominal inner SS tube size (if applicable) - cross-section configuration indicator ("F" for CFST, "D" for CFDST and "H" for CFHT). For example, "203×2-101×3-D" represents SWSSC-filled double-skin tubes with SS outer tube (D_0 =203 mm and t_0 =2 mm) and inner tube (D_i =101 mm and t_i =3 mm).

5.2.3 Test setup and instrumentation

Tensile coupon test was conducted in accordance with AS1391 (2017) to obtain the material properties of stainless steel. Two coupons were cut from each size of SS tubes and their ends were flattened to be gripped by the test machine. The loading rate was set as 2 mm/min with displacement control. A strain gauge was attached on both concave and convex sides of coupons and laser-extensometer with gauge length about 50 mm was adopted to measure the elongation.

CHS specimens (except H168×3 and H203×2) were tested in a 500 kN capacity Baldwin machine, whereas the SWSSC-filled specimens were tested in a 5000 kN Amsler machine. The axial load was

directly applied on specimens through a loading plate and the loading rate was 1 mm/min with displacement control. When casting concrete, the concrete level is about 1 mm lower than the SS tubes. One week before testing, high strength paste was used to fill this gap so that the loading plate was in contact with stainless steel tubes and concrete simultaneously. Therefore, the load was applied on both SS tubes and sandwich concrete simultaneously. Each batch of SWSSC-filled specimens was tested for 28 ± 2 days so that concrete strength caused by curing time did not change obviously within each batch. Three LVDTs were set vertically to measure the axial end shortening along the total height and the average reading was adopted as the end shortening of the specimen. Three pairs of strain gauges, one in the axial direction and one in the hoop direction, were fixed at the middle height of the outer SS tubes with equal distance. Specifically, two pairs of strain gauges with leads were embedded on the inner tubes of specimens $203\times2-101\times3$ -D and $203\times2-152\times1.6$ -D before concrete casting in order to monitor the strain development of inner SS tube. All the load, displacement, strain gauge data were recorded by a data acquisition system. The test setup and instrumentation are generally similar to those described in Chapter 3.

5.3 Experiment results

5.3.1 Tensile strength

Material properties of stainless steel obtained from tensile coupon test are summarized in Table. 5.3. The stress-strain curves of some typical coupons are plotted in Fig. 5.2, in which the initial stage strains (axial strain = $0 \sim 0.02$) are obtained from strain gauges due to the high accuracy of strain gauges whilst the later stage strains (axial strain > 0.02) are recorded using laser extensometer due to its larger measurement range. The stress-strain curve of stainless steel shows a rounded shape without yielding plateau, which is a major difference to carbon steel.

Table 5.3. Summary of tensile coupon test results

Tube size	E _o (GPa)	$f_{\rm y}$ (MPa)	f _u (MPa)	\mathcal{E}_{y}	Eu	n ^a	т	<i>E</i> _{0.2} (GPa)
50×1.6	N/A	376.5	656.8	0.40%	47.4%	7.5	3.0	22.2
50×3	N/A	228.9	562.1	0.32%	57.4%	7.5	2.4	14.1
76×1.6	193.2	398.9	732.4	0.41%	40.7%	7.5	2.9	23.3
89×3	176.2	259.2	587.8	0.34%	52.1%	7.5	2.5	15.8
101×1.6	186.6	353.3	706.0	0.39%	38.4%	7.5	2.8	21.0
101×3	192.6	226.0	656.4	0.32%	67.2%	7.5	2.2	14.0
114×3	199.3	281.2	617.8	0.35%	55.9%	7.5	2.6	17.1
152×1.6	189.0	314.5	659.8	0.37%	50.2%	7.5	2.7	18.9
168×3	190.3	281.5	615.8	0.35%	58.0%	7.5	2.6	17.1
203×2	189.6	304.0	653.1	0.36%	48.7%	7.5	2.6	18.3
Mean	189.6							
COV ^b	0.03							

^a: Adopted from Table B1(a) in AS/NZS 4673:2001; ^b: COV=coefficient of variation.



Fig. 5.2. Typical stress-strain curves from tensile coupon test and existing models

Because the tensile coupons were cut from circular tubes, the coupons were not flat. Once loaded, the coupons tended to become flat and it may affect the accuracy of the longitudinal strain gauge readings. To mitigate this issue the average reading from strain gauges on both concave and convex sides were adopted. In future study, the width of the coupon for small diameter tubes could be slightly reduced. Initial elastic modulus (E_0) is the slope of the initial linear line of the stress-strain curve. Since the curvature of the coupons for tube 50×1.6 and 50×3 was sharp, the longitudinal strain gauge readings were not reliable due to bending effects. Therefore, the E_0 of tube 50×1.6 and 50×3 is not given in Table 5.3. The average E_0 for SS is 189.6 GPa, which is close to the value of 195 GPa suggested by AS/NZS 4673:2001 (2001). Once E_0 is determined, the yield strength (f_y), which is 0.2% proof stress, and yield strain (ε_y), which is the strain at f_y , can be determined based on the stress-strain curves. The ultimate strength (f_u) and ultimate strain (ε_u) can be directly determined from the full range stress-strain curves.

5.3.2 Unfilled circular hollow sectional specimens

All the circular hollow sectional (CHS) specimens failed by the plastic local buckling and elephant foot was formed near one end, which is in agreement with the existing reports (Zhao et al. 2002; Bardi and Kyriakides 2006). The normalized stress-strain curves of CHS specimens, where the normalized stress is the stress divided by yielding stress and the strain equals to the end shortening divided by column height, are plotted in Fig. 5.3. As expected, with the increase of D_0/t_0 , ultimate normalized stress and ultimate strain decrease and the post-peak curve becomes steeper when the cross-section is slender.



Fig. 5.3. Normalized stress-strain relationships of unfilled CHS specimens

5.3.3 Concrete filled stainless steel tubes

The failure mode of Seawater and sea sand concrete (SWSSC)-filled stainless steel (SS) tubes was different from that of unfilled CHS specimens. This observation agrees to that of concrete-filled double-skin carbon steel tubes (Tao et al. 2004; Zhao et al. 2002). Images of the progressive failure of specimens are shown in Fig. 5.4. Before reaching the first peak point, no apparent SS tube buckling was observed. With the increase of axial shortening, multiple folds formed along the length of the stub columns. Generally, the failure modes of fully-filled, double-skin or double-skin tubes without inner tube were similar. Under larger deformation, specimens with lower confinement and higher void ratio tended to develop more folds.



(c) 152×1.6-50×1.6-D



(g) 168×3-89×3-D Fig. 5.4. Failure modes of some typical specimens

The load-axial strain curves of all the SWSSC-filled specimens are shown in Fig. 5.5, in which the axial strain is the axial end shortening divided by specimen height. The ultimate capacity (N_t , listed in Table 5.2) is defined as the first peak load within the 5% axial strain. If there is no peak load in this strain range (e.g., 50×3-F, 114×3-50×3-D), the load at 5% axial strain is defined as N_t . This definition was also adopted by Lam and Gardner (2008).



The load increased linearly up to about 70% of the peak load and then it tended to slow down until the peak load. After the peak load, there was a rapid load drop for most specimens and the load drop amplitude depended on several factors such as cross-section type, confinement, void ratio and tube slenderness. With further increase of axial strain, the applied load was consistent, or even increased again. For some highly confined specimens (e.g. 50×3 -F), the load-strain relationship was a strain-hardening response without any load drops. As shown in Fig. 5.5, before reaching the peak load, the load-axial strain relationships of CFST, CFDST and CFHT were very similar. The shape of the "post-

peak" curve of CFST is similar to those of CFDST with small void ratio (Fig. 5.4a-e). Furthermore, CFHT had a steeper and larger load drop than CFDST as the inward expansion of concrete cannot be restrained due to the absence of the inner SS tube (Fig. 5.4f). A comparison between Figs. 5.3 and 5.5 indicates that the in-filled concrete significantly improved the ductility in the point of view of the area covered by load-axial strain curves.

5.4 Discussions

5.4.1 Stress-strain relationship of stainless steel

In AS/NZS 4673 (2001), the stress-strain (σ - ε) relationship is expressed analytically by Ramberg-Osgood equation:

$$\varepsilon = \frac{\sigma}{E_o} + 0.002 (\frac{\sigma}{f_y})^n \tag{5.5}$$

where *n* is a constant, which is adopted as 7.5 as suggested in AS/NZS 4673 (2001). Past studies (Rasmussen 2003; Abdella 2006) indicated that the Ramberg-Osgood relation tends to overestimate the stress when the strain exceeds ε_y , particularly when *n* is low. Rasmussen (2006) proposed a two-stage stress-strain relationship for SS. Ramberg-Osgood equation is still adopted for the first stage when stress is less or equal to f_y . A new equation was proposed for the second stage (i.e. when $\sigma > f_y$):

$$\varepsilon = \frac{\sigma - f_y}{E_{0.2}} + \varepsilon_u \left(\frac{\sigma - f_y}{f_u - f_y}\right)^m + \varepsilon_y$$
(5.6)

$$E_{0.2} = \frac{E_o}{1 + 0.002n \cdot E_o / f_y}$$
(5.7)

$$m = 1 + 3.5 \frac{f_y}{f_u} \tag{5.8}$$

where $E_{0.2}$ is the tangent modulus at f_y and m is a strain hardening exponent.

A comparison between the experimental stress-strain curves and predicted curves by Ramberg-Osgood equation and Rasmussen's method is shown in Fig. 5.2. As shown in Fig. 5.2, the predicted curves show good agreement to experimental curves and the difference between the two methods is not obvious. It is necessary to mention that the stress-strain model proposed by Rasmussen will be adopted in continuous strength method (CSM) to determine the strength of unfilled SS CHS stub columns as shown in Section 5.4.2.

5.4.2 Design methods for stainless steel hollow sections

Existing design codes have provided methods to estimate the ultimate capacity of SS circular hollow sectional (CHS) stub columns under axial compression. The ratios of the predicted capacity-to-experimental capacity are listed in Table 5.4, where N_t is experimental capacity, N_y is yielding capacity (= A_0f_y), N_{AS} , N_{ASCE} , N_{EN} are the capacity determined by AS/NZS 4673 (2001), SEI/ASCE8-02 (2002) and EN 1993-1-4 (2006) respectively. It is found that all the cross-sections are fully effective except for specimens H152×1.6 (D_0/t_0 =96) and H203×2 (D_0/t_0 =102) based on SEI/ASCE 8-02 (2002). These methods offer more conservative capacity prediction for specimens with smaller D_0/t_0 . For example, the predicted capacity for H50×3, which has the smallest D_0/t_0 , is only half of its experimental capacity.

			Ŭ				<u> </u>
Specimen	$D_{\rm o}/t_{\rm o}$	$N_{\rm t}$ (kN)	$N_{\rm y}/N_{\rm t}$	$N_{\rm AS}/N_{\rm t}$	$N_{\rm ASCE}/N_{\rm t}$	$N_{\rm EN}/N_{\rm t}$	$N_{\rm CSM}/N_{\rm t}$
H50×1.6	32	91	0.96	0.96	0.96	0.96	1.04
H50×3	17	202	0.52	0.52	0.52	0.52	0.61
H76×1.6	46	144	1.08	1.08	1.08	1.08	1.16
H89×3	28	274	0.82	0.82	0.82	0.82	0.92
H101×1.6	60	184	1.02	1.02	1.02	1.02	1.09
H101×3	37	254	0.77	0.77	0.77	0.77	0.85
H114×3	41	335	0.82	0.82	0.82	0.82	0.89
H152×1.6	96	247	0.97	0.97	0.85	0.97	1.01
H168×3	52	537	0.88	0.88	0.88	0.88	0.94
H203×2	102	347	1.10	1.10	0.95	1.10	1.14
Mean			0.89	0.89	0.87	0.89	0.97
COV			0.18	0.18	0.17	0.18	0.16

Table 5.4. Predictions by existing design methods for unfilled CHS specimens

Recently, continuous strength method (CSM) was proposed by Garder and Theofanous (2008), which is more suitable for materials exhibiting a high degree of strain hardening, such as stainless steel. In CSM, the ultimate strain (ε_{LB}) is firstly determined by empirical formulas (Garder and Theofanous 2008):

$$\frac{\varepsilon_{LB}}{\varepsilon_0} = \frac{0.18}{\lambda_c^{1.24+1.70\lambda_c}}$$
(5.9)

$$\lambda_{c} = \frac{\sqrt{3(1-\nu^{2})}(D_{o} - t_{o})f_{y}}{2E_{o}t_{o}}$$
(5.10)

$$\varepsilon_0 = f_y / E_o \tag{5.11}$$

where λ_c is cross-section slenderness for CHS. The capacity corresponding to the ultimate strain can then be obtained from the stress-strain model of stainless steel (i.e., Rasmussen's model (2003)). The

ratios of predicted capacity by CSM to experimental capacity (N_{CSM}/N_t) are listed in Table 5.4, which shows higher accuracy than existing design codes.

5.4.3 Ultimate stress in concrete

It is well known that the strength of concrete in a concrete-filled tube can be enhanced due to the active confinement provided by the encased steel tubes (Gardner and Jacobson 1967; Johansson and Gylltoft 2002). In order to understand the structural behaviour of SWSSC-filled SS tubes, the stress in concrete should be investigated first. The present study approximately assumes that the stress distribution in the concrete cross-section is uniform and the stress in concrete is equal to the load carried by concrete divided by its cross-sectional area. During an experiment, it is impossible to obtain the load carried individually by concrete and SS tubes. Therefore, it is assumed that the load carried by concrete is equal to the total load subtracted by the loads carried by outer and inner SS tubes at the same strain. It is further assumed that the load-axial strain relationship of SS tube is same as that of corresponding unfilled CHS tube before reaching its peak load. After the peak load, the load carried by SS tube is assumed to be the same as the peak load since the in-filled concrete can delay or eliminate the buckling of SS tube. An example of the load distribution in a CFDST specimen is shown in Fig. 5.6, and the derived confined concrete strengths (f_{cc} ') for all SWSSC-filled specimens are listed in Table 5.2.



Fig. 5.6 Calculation of the load carried by concrete (e.g., 168×3-101×3-D)

The relationship of the confinement factor (ξ) and confined strength-to-unconfined strength ratio (f_{cc}'/f_{c}') is presented in Fig. 5.7. Generally, higher confinement factor leads to higher values of f_{cc}'/f_{c}' . Because of the influence of inner voids, CFST, CFDST and CFHT specimens exhibit different f_{cc}'/f_{c}' at a given ξ . It seems that when ξ is above 0.7, f_{cc}'/f_{c}' ratio reaches about 1.35 and a regression line for the data is shown in Fig. 5.7. The effects of confinement factor on ultimate stress of confined concrete in CFST and CFDST are similar. If there is no inner SS tube (i.e., CFHT), the confinement effectiveness is reduced, especially for the case of large void ratio, as indicated by the lower value of f_{cc}'/f_c' of CFHT than that of CFST.



Fig. 5.7. Effects of confinement factor on ultimate stress of confined concrete

The effects of void ratio (χ) on ultimate stress of confined concrete in CFDSTs are summarized in Fig. 5.8, in which χ for CFST is regarded as 1.0. The f_{cc}'/f_c' of CFDSTs with the same outer tube does not change significantly with the increase of void ratio. The confinement effectiveness for sandwich concrete in CFDST is similar to that of concrete core in CFST provided the confinement factors are the same. No clear trend can be found on the influence of void ratio on the ultimate stress of confined concrete in CFDSTs.



Fig. 5.8. Effects of void ratio on ultimate stress of confined concrete in CFDSTs

Since the effect of outer tube slenderness ratio (λ_0) is partly considered in the confinement factor, which depends on the properties of outer SS tube (i.e., D_0 , t_0 , f_{yo}) and concrete, the effect of λ_0 is not discussed separately in this study. Fig. 5.9 shows the relationship between f_{cc}'/f_c' and inner tube slenderness ratio (λ_i) for CFDSTs. No clear trend can be observed on the influence of λ_i on f_{cc}' .



Fig. 5.9. Effect of inner tube slenderness ratio on ultimate stress of confined concrete in CFDSTs

5.4.4 Post-peak behaviour

The post-peak behaviour of SWSSC-filled SS tubes differs a lot as indicated by the load-axial strain curves (Fig. 5.5). The ratio of the minimum load within 5% axial strain (after reaching peak load) to ultimate capacity (N_{min}/N_t) is adopted to quantitatively represent the post-peak behaviour, which can be regarded as a load drop index. N_{min} is the valley point of the load-axial strain curve when the axial strain ranges from the strain at peak load to 5%. It should be mentioned that for specimens without load drop within the 5% axial strain (e.g., 50×3-F), the ratio N_{min}/N_t is taken as 1.0.

The effect of confinement factor (ζ) on load drop index (N_{\min}/N_t) is presented in Fig. 5.10. Generally, higher confinement leads to higher value of N_{\min}/N_t which represents less load drop after the ultimate capacity. The highly confined specimens do not exhibit a load drop and N_{\min}/N_t is assumed to be 1. CFDST specimens with the same confinement factor show different values of N_{\min}/N_t since other parameters, such as void ratio and inner tube slenderness, could affect the post-peak behaviour as well. As shown in Fig. 5.10, double-skin specimens without inner tube (CFHT) shows more load drop than corresponding CFST or CFDSTs. A trend line (Fig. 5.10), which is obtained by regression analysis, shows an increase of N_{\min}/N_t with the increase of ζ until ζ reaches about 0.87 and then N_{\min}/N_t becomes almost a constant.

In order to assess the effect of void ratio, the specimens with similar confinement factor and similar inner tube slenderness ratio are compared. Table 5.5 summarizes three groups of specimens. It is found that specimens with different void ratios but similar ξ and λ_i exhibit similar N_{\min}/N_t . Based on the present study, the influence of void ratio on the post-peak behaviour of CFDSTs is not obvious if other parameters, such as inner tube slenderness ratio, confinement factor, do not change.



Fig. 5.10. Confinement effect on load drop index

Table 5.5. Effects of void ratio on N_{\min}/N_t of CFDSTs

Specimen	ξ	λ_{i}	χ	$N_{\rm min}/N_{\rm t}$
152×1.6-101×3-D	0.32	33	0.68	0.76
203×2-101×3-D	0.29	33	0.51	0.71
168×3-114×3-D	0.54	46	0.70	0.86
168×3-50×1.6-D	0.54	49	0.31	0.88
152×1.6-76×1.6-D	0.32	73	0.51	0.68
203×2-76×1.6-D	0.29	73	0.38	0.66

Fig. 5.11 shows the comparisons of CFDST specimens with the same confinement factor (ζ) and the same void ratio (χ) but different inner tube slenderness ratio (λ_i). Specimens with lower λ_i show higher values of N_{\min}/N_t . The relationship between N_{\min}/N_t and λ_i is shown in Fig. 5.12, in which the regression line is also presented. For a given confinement, there is almost a linear decrease trend of N_{\min}/N_t with the increase of λ_i with regardless of void ratio.



Fig. 5.11. Effect of inner tube slenderness ratio on load drop index



Fig. 5.12. Effect of inner tube slenderness ratio on load drop index

The inner tube in a CFDST can restrain the inward expansion of sandwich concrete. The importance of inner SS tube on load drop index is shown in Fig. 5.13. CFHT specimens (i.e., without inner tube) experienced 12%-51% more load drop than CFDST specimens (i.e., with inner tube). The existence of inner SS tube can greatly improve the post-peak behaviour as indicated by the higher values of N_{\min}/N_{t} .



Fig. 5.13. Effect of absence of inner SS tube on load drop index

5.4.5 Stress-strain condition of stainless steel tubes

In order to investigate the strain development of inner SS tube, both longitudinal and lateral strain gauges were attached at the mid-height of inner SS tubs of specimens $203 \times 2-101 \times 3$ -D and $203 \times 2-152 \times 1.6$ -D. The load-axial strain curves and lateral strain-axial strain curves are shown in Fig. 5.14, in which the axial and lateral strains are the average readings from strain gauges (SGs). The lateral strain (absolute value) of inner tube is lower than that of outer tube, indicating that the outer tube experienced larger lateral expansion than inner tube. The reason is that the outer tube is under the stress state of axial compression-lateral tension. On the other hand, the lateral expansion of inner SS tube due to Poisson's effect is partly compensated by the lateral pressure introduced by the inward expansion of sandwich concrete. As shown in Fig. 5.14, the dilation properties (i.e., lateral strain-

axial strain relationship) of specimens $203 \times 2 \cdot 101 \times 3 \cdot D$ ($\xi=0.29, \chi=0.51, \lambda_i=33$) and $203 \times 2 \cdot 152 \times 1.6 \cdot D$ ($\xi=0.29, \chi=0.77, \lambda_i=120$) are almost the same before reaching the ultimate capacity. Therefore, the dilation property of CFDSTs mainly depends on the outer tube (i.e., confinement factor), which is similar to fully-filled tubes. As shown in Fig. 5.14, the axial strains of inner and outer SS tubes are reasonably close.



5.4.6 Energy absorption

In the present study, energy absorption, which could represent the ductility of a specimen to some extent, is defined as the area under the load-axial strain curve (terminated at 5% axial strain) of a specimen. The energy absorptions of both CFDST specimens (E_{CFDST}) and unfilled CHS specimens (E_{CHS}) were calculated. For some unfilled outer CHS specimens, whose axial strain did not reach 5%, a straight line was extended to zero loading based on the load decreasing trend so that E_{CHS} can be calculated. The energy absorption ratio of CFDST and the corresponding outer tube (E_{CFDST}/E_{CHS}) was adopted to represent the beneficial effect of in-filled concrete to improve the energy absorption capability. Fig. 5.15 shows the relationship of E_{CFDST}/E_{CHS} and outer tube diameter-to-thickness ratio (D_0/t_0), in which the data of Zhao et al. (2002, 2010) for concrete-filled double-skin carbon steel tubes with D_0/t_0 ranging from 20 to 102 is also included. It is obvious that greater increase is achieved for specimens with more slender (larger D_0/t_0) outer tube. SS specimens shows higher value of E_{CFDST}/E_{CHS} than that of carbon steel specimens, which is probably caused by the round shape stress-strain behaviour of stainless steel.



Fig. 5.15. Comparison of energy absorption

5.5 Capacity prediction

5.5.1 Concrete fully filled stainless steel tubes

It has been demonstrated that the current design standards, which are applied for concrete-filled carbon steel tube, provide conservative prediction for the ultimate capacity of concrete-filled stainless steel tube (Lam and Gardner 2008; Uy et al. 2011). Chapter 3 modified the design method proposed by Han et al. (2005) for concrete-filled carbon steel tube and apply it to SWSSC fully filled SS tubes (CFST). The ultimate capacity can be estimated by Eqs. (12-13):

$$N_p = (A_o + A_c) f_{scy}$$
(5.12)

$$f_{scv} = (1.14 + 1.4\xi)f_c'$$
(5.13)

where f_{scy} is nominal yielding strength of composite sections. After considering all the test data in the literature (Chapters 3 and 4; Lam and Gardner 2008; Uy et al. 2011; Tam et al. 2014; Yang and Ma 2013) and those in the present chapter, the constants in Eq. (5.13) can be modified and a refined equation is proposed herein to estimate f_{scy} :

$$f_{scy} = (1+1.41\xi)f_c'$$
(5.14)

where ξ is defined by Eq. (5.3).

The relationship between f_{scy}/f_c ' and ξ of existing data and the regression lines proposed by Chapter 3 and present study (denoted as "Refined") are illustrated in Fig. 5.16. The performance of the design methods is summarized in Table 5.6, which includes all the existing experimental data of fully concrete-filled SS circular tubular stub columns. As shown in Table 5.6, the refined method offers more accurate prediction for ultimate capacity, with an average N_p/N_t ratio of 0.99 and coefficient of variation (COV) of 0.13.



Fig. 5.16. Relationship between f_{scy}/f_c ' and ξ

Table 5.6. Performance of method in Chapter 3 and newly proposed refined method for capacity prediction of CFST specimens

Data course		$f(\mathbf{MD}_{\mathbf{a}})$	$f'(\mathbf{MD}_{0})$	No. of	$N_{\rm p}/N_{\rm t}$ (method	in Chapter 3)	$N_{\rm p}/N_{\rm t}$ (Refined method)	
Data source	D_0/l_0	J _y (MPa)	J _c (IVIPa)	specimens	Mean	COV	Mean	COV
This chapter	17-102	229-399	42	10	1.14	0.04	1.07	0.04
Chapter 3	36-53	270-324	31	3	0.99	0.01	0.94	0.02
Chapter 4	17-52	226-281	36	4	1.13	0.02	1.06	0.03
Lam & Gardner 2008	19, 52	266, 412	31-65	6	1.02	0.07	0.96	0.08
Tam et al. 2014	59	340	38-42	4	1.11	0.03	1.03	0.03
Yang & Ma 2013	68	287-293	44-50	7	1.10	0.01	1.01	0.01
Uy et al. 2011	64-102	259-320	20, 30	7	0.87	0.11	0.81	0.10
All					1.06	0.13	0.99	0.13

5.5.2 Concrete-filled double-skin stainless steel tube

The discussion in Section 5.4.3 indicates that confinement factor is still the major parameter to affect the ultimate stress in confined concrete in SWSSC-filled double-skin SS tubes (CFDSTs), which is similar to fully-filled tubes, and the influences of void ratio and inner tube slenderness ratio are not obvious. Therefore, it is assumed that the ultimate capacity of CFDST is the capacity summation of the sandwich concrete with outer tube and the inner tube. The outer tube with sandwich concrete behaves similarly to CFST and the same method is adopted to estimate its ultimate capacity. The ultimate capacity of the inner tube is assumed to equal to the ultimate capacity of unfilled CHS and the continuous strength method (CSM: Gardner and Theofanous 2008) is adopted to predict its capacity.

Based on the above assumptions, the predicted capacity (N_p) of CFDST is:

$$N_{p} = (A_{o} + A_{c})f_{scy} + N_{i}$$
(5.15)

where N_i is the estimated capacity of inner SS tube determined by CSM (Eqs. (5.9-11) in Section 5.4.2) and f_{scy} is determined by Eq. (5.14). A comparison between the predicted ultimate capacities

of CFDSTs and their experimental capacities is shown in Table 5.7 and Fig. 5.17. The average N_p/N_t ratio is 0.98 with COV of 0.06, which shows a reasonable accuracy.



Fig. 5.17. Ultimate capacity prediction of CFDSTs

Table 5.7.	Ultimate	capacity	prediction	of	CFDSTs

Specimen	λ_{o}	λ_i	χ	ξ	$N_{\rm t}$ (kN)	$f_{\rm scy}$ (MPa)	N _i (kN)	$N_{\rm p}$ (kN)	$N_{\rm p}/N_{\rm t}$
101×1.6-50×1.6-D	85	49	0.50	0.59	583	77.0	95	572	0.98
101×1.6-50×3-D	85	15	0.52	0.59	634	77.0	123	592	0.93
101×3-50×1.6-D	33	49	0.52	0.64	593	80.0	95	592	1.00
101×3-50×3-D	33	15	0.53	0.64	681	80.0	123	612	0.90
114×3-50×1.6-D	46	49	0.46	0.71	804	83.8	95	790	0.98
114×3-50×3-D	46	15	0.47	0.71	891	83.8	123	809	0.91
152×1.6-50×1.6-D	120	49	0.33	0.32	1055	61.1	95	1095	1.04
152×1.6-76×1.6-D	120	73	0.51	0.32	997	61.1	166	1006	1.01
152×1.6-101×1.6-D	120	85	0.68	0.32	825	61.1	201	822	1.00
152×1.6-101×3-D	120	33	0.68	0.32	882	61.1	216	836	0.95
168×3-50×1.6-D	59	49	0.31	0.54	1569	74.2	95	1604	1.02
168×3-76×1.6-D	59	73	0.47	0.54	1470	74.2	166	1481	1.01
168×3-89×3-D	59	29	0.55	0.54	1464	74.2	252	1441	0.98
168×3-101×1.6-D	59	85	0.63	0.54	1332	74.2	201	1250	0.94
168×3-101×3-D	59	33	0.63	0.54	1354	74.2	216	1264	0.93
168×3-114×3-D	59	46	0.70	0.54	1319	74.2	300	1193	0.90
203×2-50×3-D	124	15	0.26	0.29	1653	59.3	123	1916	1.16
203×2-76×1.6-D	124	73	0.38	0.29	1658	59.3	166	1810	1.09
203×2-101×3-D	124	33	0.51	0.29	1625	59.3	216	1646	1.01
203×2-152×1.6-D	124	120	0.77	0.29	1142	59.3	249	1078	0.94
Mean									0.98
COV									0.06

5.6 Conclusions

This chapter presents an experimental study on seawater and sea sand concrete (SWSSC) filled double-skin stainless steel tubular stub columns (CHS as both outer and inner tubes) under axial compression. The unfilled CHS specimens, fully-filled specimens (CFSTs) and double-skin specimens without inner tube (CFHTs) were also tested for comparison purpose. The effects of parameters, including confinement factor (ξ), void ratio (χ), and inner tube slenderness ratio (λ_i), on the structural behaviours were investigated. Finally, design methods were proposed to estimate the ultimate capacity of SWSSC-filled tubes. The following conclusions can be made based on present study:

(1) Ultimate stress of confined concrete in a CFDST is mainly affected by its confinement factor, which is similar to fully-filled tubes. The influences of void ratio and inner tube slenderness ratio are not obvious.

(2) Confinement factor and inner tube slenderness ratio are the major parameters to affect the postpeak behaviour (i.e., load drop after peak) of CFDSTs, whilst the influence of void ratio is not obvious.

(3) Inner SS tube in CFDSTs can enhance the confinement effectiveness and reduce the load drop amplitude in comparison to CFHTs.

(4) Outer SS tube experiences larger lateral strain than inner SS tube since the stress states in them are different.

(5) Energy absorption capability is greatly enhanced by filling the concrete and the energy absorption ratio of CFDST to corresponding CHS increases with the increase of outer tube diameter-to-thickness ratio.

(6) Design method proposed in the present study offers reasonable and accurate prediction for the ultimate capacity of SWSSC-filled SS tubes.

This chapter forms part of a large research program on hybrid construction utilising SWSSC, FRP and stainless steel (SS). FRP and SS can be used as external confinement to SWSSC or as internal reinforcement bars in SWSSC. Durability is an important part of the program although this chapter deals with structural behaviour of double skin tubes at normal condition as a reference. The authors have published several papers on durability of FRP bars in SWSSC (Wang et al. 2017a, b; Wang et al. 2018; Dong et al. 2017, 2018), FRP materials in SWSSC (Guo et al. 2018), SWSSC-filled FRP tubes (Li et al. 2018c) and SWSSC beams with FRP bars (Dong et al. 2018). Research is being carried out to investigate the long-term behaviour of SWSSC-filled stainless steel tubes.

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References

Abdella, K. (2006), Inversion of a full-range stress–strain relation for stainless steel alloys, International Journal of Non-Linear Mechanics, 41, 456-63.

ANSI/AISC 360-10 (2010), Specification for structural steel buildings, Standards Australia, Sydney.

AS/NZS 4673:2001 (2001), Cold-formed stainless steel structures, Standards Australia, Sydney.

AS/NZS 5100.6 (2017), Bridge design - steel and composite construction, Standards Australia, Sydney.

AS 1012.9:2014 (2014), Methods of testing concrete - compressive strength tests - concrete, mortar and grout specimens, Standards Australia, Sydney.

AS 1391-2007(R2017), Metallic materials - tensile testing at ambient temperature, Standards Australia, Sydney.

AS 4100-1998(R2016), Steel structures, Standards Australia, Sydney.

Bardi, F. and Kyriakides, S. (2006), Plastic buckling of circular tubes under axial compression—part I: experiments, International Journal of Mechanical Sciences, 48, 830-41.

Chen, G.M., He, Z.B., Jiang, T., Chen, J.F. and Teng, J.G. (2017), Axial compression tests on FRPconfined seawater/sea-sand concrete, 6th Asia-Pacific Conference on FRP in Structures.

Dong, Z., Wu, G., Zhao, X.-L. and Wang, Z.-K. (2017), A refined prediction method for the long-term performance of BFRP bars serviced in field environments, Construction and Building Materials, 155, 1072-80.

Dong, Z.Q., Wu, G., Zhao, X.L., Lian, J.L. (2018), Long-term bond durability of fiber-reinforced polymer bars embedded in seawater sea-sand concrete under ocean environments, Journal of Composites for Construction, 22(5), 04018042.

EN1993-1-4 (2006), Eurocode 3 - design of steel structures - part 1-4: general rules - supplementary rules for stainless steels.

EN1994-1-1 (2004), Eurocode 4: Design of composite steel and concrete structures - part 1-1: general rules and rules for buildings.

Gardner, L., and Theofanous, M. (2008), Discrete and continuous treatment of local buckling in stainless steel elements, Journal of Constructional Steel Research, 64, 1207-16.

Gardner, N.J. and Jacobson, E.R. (1967), Structural behavior of concrete filled steel tubes, Journal Proceedings, 404-13.

Guo, F., Al-Saadi, S., Singh Raman, R.S. and Zhao, X. (2018), Durability of fiber reinforced polymer (FRP) in simulated seawater sea sand concrete (SWSSC) environment, Corrosion Science, 141, 1-13.

Han, L.-H., Ren, Q.-X. and Li, W. (2011), Tests on stub stainless steel–concrete–carbon steel doubleskin tubular (DST) columns, Journal of Constructional Steel Research, 67, 437-52.

Han, L.-H., Xu, C.-Y. and Tao, Z. (2018), Performance of concrete filled stainless steel tubular (CFSST) columns and joints: Summary of recent research, Journal of Constructional Steel Research. 2018. (in press)

Han, L.-H., Yao, G.-H. and Zhao, X.-L. (2005), Tests and calculations for hollow structural steel (HSS) stub columns filled with self-consolidating concrete (SCC), Journal of Constructional Steel Research,61, 1241-69.

Han, T.H., Stallings, J.M. and Kang, Y.J. (2010), Nonlinear concrete model for double-skinned composite tubular columns, Construction and Building Materials, 24, 2542-53.

Huang, H., Han, L.-H., Tao, Z. and Zhao X-L. (2010), Analytical behaviour of concrete-filled double skin steel tubular (CFDST) stub columns, Journal of Constructional Steel Research, 66, 542-55.

Johansson, M. and Gylltoft, K. (2002), Mechanical behavior of circular steel–concrete composite stub columns, Journal of Structural Engineering, 128, 1073-81.

Lam, D. and Gardner, L. (2008), Structural design of stainless steel concrete filled columns, Journal of Constructional Steel Research, 64, 1275-82.

Li, Y.L., Zhao, X.L., Raman Singh, R.K. and Al-Saadi, S. (2016a), Experimental study on seawater and sea sand concrete filled GFRP and stainless steel tubular stub columns, Thin-Walled Structures, 106, 390-406.

Li, Y.L., Zhao, X.L., Raman Singh, R.K. and Al-Saadi, S. (2016b), Tests on seawater and sea sand concrete-filled CFRP, BFRP and stainless steel tubular stub columns, Thin-Walled Structures, 108, 163-84.

Li, Y.L., Zhao, X.L., Raman Singh, R.K. and Al-Saadi, S. (2018a), Thermal and mechanical properties of alkali-activated slag paste, mortar and concrete utilising seawater and sea sand, Construction and Building Materials, 159, 704-24.

Li, Y.L., Teng, J.G., Zhao, X.L. and Singh Raman, R.K. (2018b), Theoretical model for seawater and sea sand concrete-filled circular FRP tubular stub columns under axial compression, Engineering Structures, 160, 71-84.

Li, Y.L., Zhao, X.L. and Singh Raman, R.K. (2018c), Durability of seawater and sea sand concrete-filled FRP tubes in artificial seawater, Construction and Building Materials. (under review)

Nishida, T., Otsuki, N., Ohara, H., Garba-Say, Z.M. and Nagata, T. (2015), Some considerations for applicability of seawater as mixing water in concrete, Journal of Materials in Civil Engineering, 7, B4014004-1-7.

Rasmussen, K.J. (2003), Full-range stress-strain curves for stainless steel alloys, Journal of Constructional Steel Research, 59, 47-61.

SEI/ASCE8-02 (2002). Specification for the design of cold-formed stainless steel structural members.
Shams, M. and Saadeghvaziri, M.A. (1997), State of the art of concrete-filled steel tubular columns, ACI Structural Journal, 94, 558-71.

Tam, V.W., Wang, Z.-B. and Tao Z. (2014), Behaviour of recycled aggregate concrete filled stainless steel stub columns, Materials and Structures, 47, 293-310.

Tao, Z., Han, L.-H. and Zhao, X.-L. (2004), Behaviour of concrete-filled double skin (CHS inner and CHS outer) steel tubular stub columns and beam-columns, Journal of Constructional Steel Research, 60, 1129-58.

Teng, J.G., Yu, T., Dai, J.G. and Chen, G.M. (2011), FRP composites in new construction: current status and opportunities, in: Proceedings of 7th National Conference on FRP Composites in Infrastructure (Supplementary Issue of Industrial Construction), keynote presentation.

Teng, J.G. (2014), Performance enhancement of structures through the use of fibre-reinforced polymer (FRP) composites, in: Proceedings of 23rd Australasian Conference on the Mechanics of Structures and Materials (ACMSM23), Lismore, Australia, keynote presentation.

Uenaka, K., Kitoh, H. and Sonoda, K. (2010), Concrete filled double skin circular stub columns under compression, Thin-Walled Structures, 48, 19-24.

Uy, B., Tao, Z. and Han, L.-H. (2011), Behaviour of short and slender concrete-filled stainless steel tubular columns, Journal of Constructional Steel Research, 67, 360-78.

Wang, Z., Zhao, X.-L., Xian, G., Wu, G., Singh Raman, R.S. and Al-Saadi, S. (2018), Effect of sustained load and seawater and sea sand concrete environment on durability of basalt-and glass-fibre reinforced polymer (B/GFRP) bars, Corrosion Science, 138, 200-18.

Wang, Z., Zhao, X.-L., Xian, G., Wu, G., Singh Raman, R.S., Al-Saadi, S. et al. (2017a), Long-term durability of basalt-and glass-fibre reinforced polymer (BFRP/GFRP) bars in seawater and sea sand concrete environment, Construction and Building Materials, 139, 467-89.

Wang, Z., Zhao, X.-L., Xian, G., Wu, G., Singh Raman, R.S. and Al-Saadi, S. (2017b), Durability study on interlaminar shear behaviour of basalt-, glass- and carbon-fibre reinforced polymer (B/G/CFRP) bars in sea water sea sand concrete environment, Construction and Building Materials, 156, 985-1004.

Wei, S., Mau, S., Vipulanandan, C. and Mantrala, S. (1995a), Performance of new sandwich tube under axial loading: Analysis, Journal of Structural Engineering, 121, 1815-21.

Wei, S., Mau, S., Vipulanandan, C. and Mantrala, S. (1995b), Performance of new sandwich tube under axial loading: Experiment, Journal of Structural Engineering, 121, 1806-14.

Xiao, J., Qiang, C., Nanni, A. and Zhang, K. (2017), Use of sea-sand and seawater in concrete construction: current status and future opportunities, Construction and Building Materials, 155, 1101-11.

Yang, Y.-F., Han, L.-H. and Sun, B.-H. (2012), Experimental behaviour of partially loaded concrete filled double-skin steel tube (CFDST) sections, Journal of Constructional Steel Research, 71, 63-73.

Yang, Y.-F. and Ma, G.-L. (2013), Experimental behaviour of recycled aggregate concrete filled stainless steel tube stub columns and beams, Thin-Walled Structures, 66, 62-75.

Zhao, X.-L., Grzebieta, R. and Elchalakani, M. (2002), Tests of concrete-filled double skin CHS composite stub columns, Steel and Composite Structures, 2, 129-46.

Zhao, X.L., Han, L.H. and Lu, H. (2010), Concrete-filled tubular members and connections, CRC Press, London.

Zhao, X.L. and Han, L.H. (2006), Double skin composite construction, Progress in Structural Engineering and Materials, 8, 93-102.

Zhao, X.-L., Tong, L.-W. and Wang, X.-Y. (2010), CFDST stub columns subjected to large deformation axial loading, Engineering Structures, 32, 692-703.

Chapter 6

Theoretical model for seawater and sea sand concrete-filled circular FRP tubular stub columns under axial compression

Abstract

The use of FRP with seawater and sea sand concrete (SWSSC) holds great potential for marine and coastal infrastructure, and concrete-filled FRP tubular columns are among the attractive forms of structural members for such applications. This chapter presents a theoretical model for the compressive behaviour of seawater and sea sand concrete-filled circular FRP tubular stub columns. FRP tubes can be manufactured to possess considerable strength and stiffness in the longitudinal direction, so the behaviour of concrete-filled FRP tubes differed substantially from that of concrete columns with an FRP wrap (also referred to as "concrete-filled FRP wraps") which commonly contains fibres only in the hoop direction. Many theoretical models have been proposed for concretefilled FRP wraps, but very limited work has been conducted on the theoretical modelling of concretefilled FRP tubes. In the present study, an existing dilation model for concrete-filled FRP wraps is combined with a biaxial stress analysis of the FRP tube so that the effect of the Poisson's ratio of the FRP tube is properly accounted for. In order to predict the buckling of the FRP tube, a maximum strain buckling failure criterion is proposed and is shown to be in reasonable agreement with the experimental results. Moreover, the load carried by the FRP tube is studied, and a simplified model is proposed to determine the load shared by the FRP tube during the entire loading process. Finally, a theoretical model for SWSSC-filled FRP tubular columns is proposed, in which the behaviour of both the concrete and the FRP tube as well as their interactions are explicitly modelled (i.e., an analysis-oriented model). The proposed model gives reasonably close predictions of the existing experimental data.

Keywords

Concrete-filled FRP tube, confinement, dilation, theoretical model, seawater and sea sand concrete, axial compression

Nomenclature

- A_c = cross-sectional area of concrete
- A_f = cross-sectional area of FRP tube
- D_i = diameter of concrete core (= D_0 -2 t_0)
- D_o = outer diameter of FRP tube
- E_c = elastic modulus of unconfined concrete
- E_h = elastic modulus of FRP tube in hoop direction
- E_{hsec} = secant modulus of FRP tube in hoop direction
- E_l = elastic modulus of FRP tube in longitudinal direction
- f_{co}' = unconfined concrete strength
- f_{cc}^{*} = peak stress of confined concrete

f_l	=	confining pressure
fuh	=	hoop tensile strength of FRP tube (by split-disk test)
ful,c	=	longitudinal compressive strength of FRP tube (by compressive test on short tube)
$f_{ul,t}$	=	longitudinal tensile strength of FRP tube (by tensile coupon test)
Ι	=	number of incremental steps
k_1	=	factor 1
k_2	=	factor 2
k3	=	factor 3
Ν	=	load carried by column
N_f	=	load carried by FRP tube
N _{fu}	=	ultimate load of FRP tube
N_t	=	experimental ultimate load
to	=	thickness of FRP tube
$\varDelta \varepsilon_h$	=	hoop strain increment
\mathcal{E}_{c}	=	axial strain
$\mathcal{E}_{c,SG}$	=	axial strain (from strain gauge)
\mathcal{E}_{cb}	=	axial strain at tube buckling (predicted)
$\mathcal{E}_{cb,t}$	=	axial strain at tube buckling (experiment)
$oldsymbol{\mathcal{E}}^{*}_{cc}$	=	axial strain at confined concrete peak stress f_{cc}^{*}
\mathcal{E}_{co}	=	axial strain at unconfined concrete strength f_c '
\mathcal{E}_{CU}	=	ultimate axial strain
\mathcal{E}_{fu}	=	axial strain at ultimate load of FRP tube $N_{\rm fu}$
\mathcal{E}_h	=	hoop strain
\mathcal{E}_{hb}	=	hoop strain at tube buckling
\mathcal{E}_{hu}	=	ultimate hoop strain of FRP tube
\mathcal{E}_l	=	longitudinal strain in FRP tube
\mathcal{E}_{lat}	=	lateral strain
η	=	reduction factor for FRP tube
ρ_K	=	confinement stiffness
σ_h	=	hoop stress in FRP tube
σ_l	=	longitudinal stress in FRP tube
v_h	=	Poisson's ratio (hoop tension)
v_l	=	Poisson's ratio (longitudinal compression)

6.1 Introduction

In recent decades, fiber reinforced polymer (FRP) has been increasingly used in civil engineering due to its high strength-to-weight ratio and desirable durability performance. One of the applications is in concrete-filled FRP tubular members, in which the FRP tube, with appropriate fibre orientations, can be used to provide strength and stiffness in both the longitudinal and the hoop directions. In such columns, the FRP tube acts as the stay-in-place formwork for concrete casting and provides confinement to the core concrete to enhance its strength and ductility, in addition to serving as the longitudinal and shear reinforcement (Fam et al. 2003; Mirmiran 2006). Due to the absence of steel

in such columns, seawater and sea sand concrete (SWSSC) can be used instead of ordinary concrete, leading to FRP-SWSSC hybrid systems, an attractive concept with great potential that was first proposed by the second author for marine and coastal infrastructure (Teng 2014; Teng et al. 2011). The usage of SWSSC can greatly reduce the consumption of fresh water and river sand, which considerably alleviates the resource shortage problem and environmental burden created by marine infrastructure development. This chapter forms part of a large research program currently in progress at Monash University (Li et al. 2016a, b, 2018; Wang et al. 2017a, b) in collaboration with The Hong Kong Polytechnic University, Southeast University and Harbin Institute of Technology. To facilitate the application of concrete-filled FRP tubular columns in engineering practice, an accurate theoretical model for predicting their behaviour is required.

The key to predicting the behaviour of concrete-filled FRP tubes is the prediction of response of concrete confined with an FRP tube. It is well understood that the confinement provided by an FRP wrap/tube is passive in nature (Teng and Lam 2004). Passive confinement refers to situations where the confining pressure increases continuously with the lateral strain of concrete, while active confinement refers to situations where the confining pressure is constant throughout the axial loading process. When the concrete in an FRP wrap (which has negligible longitudinal stiffness/strength) is subjected to axial compression, its lateral expansion is confined by the FRP wrap which is subjected to hoop tension. The confining pressure increases continuously due to the linear elastic stress-strain behaviour of FRP, which is different from the confinement mechanism of concrete-filled steel tube; in the latter, the confining pressure is reasonably constant after the yielding of steel, so the confinement mechanism is close to that of active confinement. The confinement mechanism of concrete-filled FRP tubes is similar to that of concrete confined with an FRP wrap except that the FRP tube, whose longitudinal stiffness/strength is significant, is in a biaxial stress state. Extensive research (Ozbakkaloglu et al. 2013) has been conducted on FRP-confined circular concrete columns [or simply referred to as "FRP-confined concrete" as the concrete in a circular section is under (nominally) uniform confinement] and many stress-strain models have been proposed. These models are generally classified as design-oriented models, which are in closed-form expressions and are easy to use in design, or analysis-oriented models, which employ an incremental numerical procedure (Teng and Lam 2004; Ozbakkaloglu et al. 2013). Analysis-oriented models for FRP-confined concrete, that are more versatile and powerful than design-oriented models, can potentially be applied to concrete confined by any material (Teng et al. 2007).

Most of the existing analysis-oriented models for FRP-confined concrete (Binici 2005; Chun and Park 2002; Fam and Rizkalla 2001; Harries and Kharel 2002; Jang and Teng 2007; Marques et al. 2004; Mirmiran and Shahawy 1997; Spoelstra and Monti 1999) were based on an active confinement

base model, and the model of Mander et al. (1988), with or without modifications, has been widely adopted as this base model. It has been commonly assumed that the prior stress-path of the confined concrete does not affect its subsequent stress-strain behaviour. Among the existing analysis-oriented stress-strain models for FRP-confined concrete, only Fam and Rizkalla's model (2001) was developed explicitly for concrete-filled FRP tubes in which the biaxial behaviour of the FRP tube is considered; however, all existing analysis-oriented stress-strain models can be easily adapted to model concrete-filled FRP tubes if the FRP tube does not suffer from local buckling failure.

Based on the authors' experimental observations (Chapters 3 and 4), the structural behaviour of SWSSC-filled FRP tubes under axial compression is different from that of concrete-filled FRP wraps: (a) the FRP tube can buckle much earlier than rupture failure due to hoop tension; (b) the dilation behaviour of concrete-filled FRP tubes is affected by the Poisson's effect of FRP tube; and (c) the FRP tube makes a significant contribution to the load-carrying capacity and this contribution should not be ignored. Obviously, as the existing analysis-oriented models for FRP-confined concrete, except Fam and Rizkalla's model (2001), were established for ordinary concrete-filled FRP wraps, these models do not account for the factors listed as (a)-(c) in the preceding sentence. Even Fam and Rizkalla's model (2001) accounts for only two of the factors, and it does not consider the effect of buckling and post-buckling behaviour of FRP tube. Furthermore, it is found that Fam and Rizkalla's model (2001) gives a much lower ultimate load for specimens in the authors' experiments (Chapters 3 and 4). if the biaxial failure envelope suggested in Fam and Rizkalla (2001) is adopted, resulting in inaccuracy in the predicted load-axial strain curves. Therefore, for this simulation exercise on SWSSC-filled FRP tubes, a suitable model is required, especially when the longitudinal strength and stiffness of the FRP tube are comparable to those in the hoop direction.

This chapter presents a theoretical model for the compressive behaviour of SWSSC-filled FRP circular tubes, in which an analysis-oriented stress-axial strain model is employed to depict the behaviour of confined SWSSC. Compared to existing analysis-oriented stress-strain models for FRP-confined concrete, the proposed model features improvements mainly in the following aspects: (a) the analysis-oriented stress-strain model presented in Jiang and Teng (2007), which was modified from an earlier model proposed by the same group (Teng et al. 2007), is adapted to consider the effect of biaxial behaviour of FRP tube on the confinement of core concrete; (b) the occurrence of tube buckling and post-buckling behaviour are represented; (c) the contribution of the FRP tube to the axial load-carrying capacity is included. In developing the theoretical model, it is assumed that the behaviour of SWSSC under confinement is the same as that of ordinary concrete, provided the two concretes have the same compressive strength. The limited existing evidence supports this assumption

(Chapter 2; Chen et al. 2017a, b), whose validity will be assessed as part of the presents study. Finally, the proposed model is verified with the experimental results of the authors and other studies.

6.2 Experimental data

An experimental database of concrete-filled FRP tubes under axial compression was employed in the present study to support the development of a theoretical model. This database included 12 specimens with multiple fiber directions (denoted by $[\pm 15/\pm 45\pm75]$, Chapters 3 and 4) and 3 specimens with (almost) unidirectional fibers (denoted by $[\pm 89]$, newly tested specimens). Seawater and sea sand concrete (SWSSC) of normal strength was used in these specimens. Details of the specimens are summarized in Table 6.1. The material properties of FRP are summarized in Table 6.2 and the typical stress-strain curves of FRP in the hoop direction are shown in Fig. 6.1. The longitudinal compressive strength is much lower than the corresponding tensile strength due mainly to the micro-buckling of fibers. As shown in Fig. 6.1, the stress-strain curves of FRP tubes with fibers exclusively oriented in the hoop direction (i.e. 89°) show pure linearity, while slight nonlinearity is observed for FRP tubes with multiple fiber directions. The secant modulus, which is the ratio of ultimate stress to ultimate strain, is reported in Table 6.2. The secant modulus (E_{hsec}) is a little lower than the elastic modulus (E_{h}) determined according to ASTM D3039/D3039M-14 (2014).

Specimen	Stacking sequence ^a	FRP type	D _o (mm)	t _o (mm)	f _{co} ' (MPa)	$N_{\rm t}$ (kN)
G50-C	$[\pm 15/\pm 45/\pm 75]$	GFRP	51.1	3.07	29.8	244
G101-C	$[\pm 15/\pm 45/\pm 75]$	GFRP	100.1	3.13	29.8	670
G114-C	$[\pm 15/\pm 45/\pm 75]$	GFRP	115.2	3.13	29.8	813
G165-C	$[\pm 15/\pm 45/\pm 75]$	GFRP	158.2	3.14	29.8	1336
C50-C	$[\pm 15/\pm 45/\pm 75]$	CFRP	50.5	2.81	35.8	388
C101-C	$[\pm 15/\pm 45/\pm 75]$	CFRP	99.9	2.81	35.8	1131
C114-C	$[\pm 15/\pm 45/\pm 75]$	CFRP	114.6	2.75	35.8	1416
C165-C	$[\pm 15/\pm 45/\pm 75]$	CFRP	158.1	2.79	35.8	2372
B50-C	$[\pm 15/\pm 45/\pm 75]$	BFRP	50.0	2.71	32.8	259
B101-C	$[\pm 15/\pm 45/\pm 75]$	BFRP	100.0	2.92	32.8	656
B114-C	$[\pm 15/\pm 45/\pm 75]$	BFRP	114.5	2.78	32.8	825
B165-C	$[\pm 15/\pm 45/\pm 75]$	BFRP	157.7	2.71	32.8	1345
G101-89-C	[±89]	GFRP	98.8	2.17	42.8	1081
C101-89-C	[±89]	CFRP	98.6	2.08	42.8	1837
B101-89-C	[±89]	BFRP	98.4	1.96	42.8	1094

Table 6.1. Specimen details

^a: Angles with respect to longitudinal direction (in degrees).

		Hoop tension				Axial c	ompressi	Axial tension	
Stacking sequence	FRP type	f _{uh} (MPa)	E _h (GPa)	$v_{ m h}$	E _{hsec} (GPa)	f _{ul,c} (MPa)	E ₁ (GPa)	Vl	<i>f</i> _{ul,t} (MPa)
	GFRP	308.8	25.2	0.44	21.1	194.7	21.5	0.27	217.6
$[\pm 15/\pm 45/\pm 75]$	CFRP	592.8	66.7	0.52	59.8	162.9	40.0	0.26	242.9
	BFRP	331.1	24.3	0.30	22.6	113.6	23.9	0.29	124.0
	GFRP	787.8	49.7	N/A	N/A	61.3	13.6	0.06	N/A
[±89]	CFRP	1658.4	162.5	N/A	N/A	99.1	12.2	0.05	N/A
	BFRP	935.5	61.0	N/A	N/A	33.2	23.2	0.11	N/A

Table 6.2. Material properties of FRP



All the specimens failed by FRP tube rupture in the hoop direction, accompanied by a sudden loss of load-carrying capacity. Buckling of FRP tubes with multiple fiber directions under axial compression occurred much earlier than tube rupture, but the specimen could still carry an increasing load after tube buckling. Buckling of the FRP tube caused a small sudden drop of applied load with a loud noise and obvious change of tube appearance. After the first appearance of buckling deformation in an FRP tube, more buckles appeared at different locations and this gradual process lasted until tube rupture (either the first appearance of a buckle or the gradual process of buckle appearance is simply referred to as "buckling" when such use does not cause confusion). By contrast, tube buckling was not observed for concrete-filled FRP tubes with fibers exclusively oriented in the hoop direction; that is, these specimens behaved similar to concrete confined by an FRP wrap (Ozbakkaloglu and Lim 2013; Wu et al. 2006; Yu et al. 2015; Zhang et al. 2014). It is worth mentioning that the buckling of FRP tube has not been given much attention in previous studies on concrete-filled FRP tubes as all these studies focussed on FRP tubes having fibers oriented close to the hoop direction. The load-axial strain curves of specimens are shown in Fig. 6.2, in which the occurrence of the first buckle of an FRP tube is highlighted by a hollow circular symbol. The load-axial strain curves of all the specimens display an ascending bilinear shape, indicating that the concrete was sufficiently confined (Lam and Teng 2003). As shown in Fig. 6.2(a), the buckling of FRP tube caused a slight drop of applied load. The load drop of GFRP tube specimens is more obvious than that of CFRP and BFRP specimens due to the higher longitudinal strength-to-hoop strength ratio ($f_{ul,c}/f_{uh}$) of 0.63 in comparison to the ratios of 0.28 and 0.34 for CFRP and BFRP tubes respectively. The shapes of load-strain curves are similar for both concrete-filled FRP tubes with multiple fiber directions and those with only hoop fibers. It should be noted that the buckling of FRP (CFRP, GFRP and BFRP) tubes all occurred when the axial strain reached around 0.01 to 0.015. This is true for both unfilled FRP tubes and filled FRP tubes as reported in Chapters 3 and 4. For an unfilled FRP tube, once buckling occurs, the load-carrying capacity of the tube reduces rapidly. However, an FRP tube supported by infilled concrete can still carry load even after the occurrence of buckling.



6.3 Analysis-oriented model

6.3.1 Axial strains

The axial strain is an important variable in an analysis-oriented model. Different methods (Zhang et al. 2014; Ozbakkaloglu et al. 2016) have been adopted by researchers to measure the axial strain, which can be classified as deformation measurement by LVDTs and strain measurement by strain gauges (SGs). LVDTs can measure the overall deformation over the whole height of a specimen, and the axial strain (ε_c) is found as the deformation divided by the gauge length (e.g. specimen height). The axial strain determined by LVDTs is helpful to evaluate the overall deformation characteristics of a specimen, but it may contain strains caused by tube local buckling. By contrast, strain gauges can measure the "localized" strain ($\varepsilon_{c,SG}$) at a specific location, which is closer to the real strain on the FRP tube. However, the strain gauge is likely to be damaged by tube buckling or the exhaustion of its measurement range during a test. Past studies (Chapter 3; Zhang et al. 2014) have indicated that the axial strain determined by LVDTs is larger than that obtained by strain gauges.

In the present study, the axial strain determined by LVDTs is adopted during the validation of the proposed theoretical model. The axial strain determined by strain gauges is used to verify the stresses in the FRP tube and to develop the failure criterion for FRP tube buckling. Therefore, the relationship between the two kinds of axial strain should be clarified. The two axial strain values can be related to each other as follows based on the test results:

$$\varepsilon_{c,SG} = k_1 \varepsilon_c \tag{6.1}$$

The relationships between axial strains obtained from LVDTs and those from SGs of the tested specimens are shown in Fig. 6.3, from which the best-fit value for k_1 for the above linear equation can be found: $k_1 = 0.8$ for SWSSC-filled FRP tubes with multiple fiber directions and 0.9 for tubes with only hoop direction fibers. It should be emphasized that the values for k_1 proposed here are derived from the authors' experiments (Chapters 3 and 4) and cannot be directly applied to other studies. If the relationship between the axial strains obtained from LVDTs and SGs is not available, the k_1 is conservatively set as 1.0 which may lead to a slight lower prediction for the buckling load of FRP tube.



6.3.2 Dilation properties

6.3.2.1 Existing dilation models

All the analysis-oriented models are built on the consideration of interaction between the external confining FRP wrap/tube and the core concrete, and this interaction can be established via a dilation model (i.e. lateral strain-axial strain relationship). The lateral strain-axial strain curves of both unconfined concrete and actively confined concrete display an exponential shape, with the slope of the second-portion of the curve for FRP-confined concrete remaining stable due to the gradual increase in confining pressure (Sfer et al. 2002; Smith et al. 1989; Lim and Ozbakkaloglu 2014). The lateral strain-axial strain relationship has been established via many approaches, either implicitly

(Binici 2005; Chun and Park 2002; Fam and Rizkalla 2001; Marques et al. 2004; Spoelstra and Monti 1999; Albanesi et al. 2007) or explicitly (Teng et al. 2007; Harries and Kharel 2002; Mirmiran and Shahawy 1997; Lim and Ozbakkaloglu 2014). Most of these relationships were derived from tests on concrete confined by an FRP wrap, except for Fam and Rizkalla's model (2002), which was derived with consideration of Poisson's effect of the FRP tube.

A comparison of the predictions by some typical dilation models with an experimental lateral strainaxial strain curve of SWSSC-filled FRP tube with fibers exclusively oriented in the hoop direction is shown in Fig. 6.4. These models exhibit different degrees of accuracy, and Teng et al's dilation model (Teng et al. 2007) is adopted in the present study due to its accuracy as well as its explicit form and simplicity. Jiang and Teng's study (2007) indicated that the accuracy of the predicted stress-strain curve of FRP-confined concrete is not sensitive to the details of the lateral strain-axial strain curve provided the ultimate point is precisely predicted and the overall trend of the curve is well captured. The comparison in Fig. 6.4 also indicates that the use of seawater and sea sand in casting the test specimens did not significantly change the dilation behaviour of the concrete.



Fig. 6.4. Comparison of model predictions with dilation curve of specimens

6.3.2.2 Dilation model for concrete in an FRP tube

The major difference between an FRP tube and an FRP wrap is that the FRP tube has substantial strength and stiffness in the longitudinal direction, and as a result, the FRP tube is in a biaxial state of stresses and is significantly affected by the Poisson's ratio. The dilation curves of SWSSC-filled FRP tubes with the layout of $[\pm 15/\pm 45/\pm 75]$ (regarded as tubes) and $[\pm 89]$ (regarded as wraps) are compared in Fig. 6.5. In this chapter perfect deformation compatibility is assumed between the concrete and the FRP tube, the hoop strain (ε_h) in the FRP tube is thus taken to be the same as the lateral strain (ε_{tat}) of the concrete. As shown in Fig. 6.5, the trends of the curves are similar except the initial portion, in which the concrete-filled FRP tube has a steeper slope due to the higher Poisson's ratio of an FRP tube to separate from the core concrete before the unstable expansion of concrete and the assumption of $\varepsilon_h = \varepsilon_{tat}$ may not be correct. However, this inaccuracy during the initial stage. Therefore a dilation model developed for concrete-filled FRP wraps is also suitable for concrete-filled FRP tubes if the Poisson's effect and the biaxial stress state are accounted for in the model.



Fig. 6.5. Dilation curves of concrete-filled FRP tubes and wraps

In the present study, Teng et al's dilation model (Teng et al. 2007) is combined with equations for the biaxial behaviour of the FRP tube to predict the lateral strain-axial strain relationship of SWSSC-filled FRP tubes. The equations in Teng et al's dilation model are as follows:

$$\frac{\varepsilon_c}{\varepsilon_{co}} = 0.85\{[1+0.75(\frac{-\varepsilon_{lat}}{\varepsilon_{co}})]^{0.7} - \exp[-7 \cdot (\frac{-\varepsilon_{lat}}{\varepsilon_{co}})]\} \cdot (1+8\frac{f_l}{f_{co}'})$$
(6.2)

$$f_l = \frac{\sigma_h t_o}{D_l / 2} \tag{6.3}$$

$$\sigma_h = E_h \varepsilon_h \tag{6.4}$$

where ε_{co} is 0.000937 $\sqrt[4]{f_{co}}$ (with f_{co} ' in MPa) as suggested by Popovics (1973). As aforementioned, the FRP tube is in a biaxial stress state and the FRP tube can be approximated as an orthotropic elastic membrane (Xie et al. 2017):

$$\begin{cases} \mathcal{E}_{c,SG} \\ \mathcal{E}_{h} \end{cases} = \begin{bmatrix} \frac{1}{E_{l}} & \frac{V_{h}}{E_{h}} \\ \frac{V_{l}}{E_{l}} & \frac{1}{E_{h}} \end{bmatrix} \begin{cases} \sigma_{l} \\ \sigma_{h} \end{cases}$$
(6.5)

where $\varepsilon_{c,SG}$ is the local axial strain in the FRP tube (= $k_1 \varepsilon_c$) and ε_h is the hoop strain in the FRP tube (= ε_{lat}). The hoop stress (σ_h) in Eq. (6.5) can be rewritten as:

$$\sigma_h = \frac{E_h(\varepsilon_h - v_l k_1 \varepsilon_c)}{1 - v_l v_h} \tag{6.6}$$

The hoop strain-axial strain relationship of concrete-filled FRP tube can be established by incorporating Eq. (6.6) into Eqs. (6.2-3):

$$\varepsilon_c = \frac{(\Phi - \Phi K \varepsilon_h) \varepsilon_{co}}{1 - \Phi K k_1 \nu_l \varepsilon_{co}}$$
(6.7)

where $\Phi = 0.85\{[1+0.75(\frac{-\varepsilon_h}{\varepsilon_{co}})]^{0.7} - \exp[-7 \cdot (\frac{-\varepsilon_h}{\varepsilon_{co}})]\}$ and $K = \frac{8}{f_{co}} \cdot \frac{E_h t_o}{(D_i / 2) \cdot (v_l v_h - 1)}$.

An FRP tube can buckle well before tube rupture as observed in the experiments (Section 2). After tube buckling, the longitudinal stiffness of the FRP tube ($\varepsilon_{c,SG}$) is much reduced and Eq. (6.5) is no longer applicable. Because of the difficulty in accurately determining the "real" axial response of post-buckled FRP tube and the fact that the axial stress in the FRP tube is largely released due to tube buckling, it is assumed that the post-buckled FRP tube behaves as an FRP wrap and Teng et al.'s dilation model (2007) can be directly used. In conclusion, Eq. (6.7) is proposed for SWSSC-filled FRP tubes before tube buckling, and Eq. (6.2) is adopted as the hoop strain-axial strain relationship after tube buckling. It is further proposed that a horizontal line be used to connect the curves obtained by Eq. (6.7) and Eq. (6.2) as shown in Fig. 6.6 to approximate the transition process. The prediction of the first occurrence of local buckling of FRP tube will be discussed in Section 3.3.1.



Fig. 6.6. Conceptual illustration of dilation model for concrete-filled FRP tubes

Comparisons between the predicted lateral strain-axial strain curves and the experimental curves are shown in Fig. 6.7, in which the first occurrence of buckling of FRP tube is also marked. In some specimens, the hoop strain gauges were damaged by the buckling of FRP tube and the experimental curves shown end before the ultimate load capacity is reached. As shown in Fig. 6.7, the predictions are in good agreement with the experimental data, indicating that the proposed dilation model (i.e. Eq. (6.7)) can be used to form part of a theoretical model for SWSSC-filled FRP tubes under axial compression.







Fig. 6.7. Verification of dilation model

6.3.3 Behaviour of FRP tube

6.3.3.1 Buckling of FRP tube

Since the FRP tube and the core concrete are subjected to simultaneous axial compression, the FRP tube is in a biaxial stress state (axial compression plus hoop tension). The commonly used failure theories for FRP laminates under combined stresses include the maximum stress theory, the maximum strain theory and the Tsai-Wu theory (Daniel and Ishai 2006). Among them, the maximum stress theory ignores the Poisson's effect while the Tsai-Wu theory needs the properties of unidirectional lamina, which are difficult to obtain for FRP tubes. After assessing the suitability of these theories, the maximum strain theory was eventually chosen in the present study due to its simplicity and acceptable accuracy.

Besides the biaxial stress state, the interaction between the FRP tube and the core concrete, which leads to the confining pressure, should also be considered in developing the failure criterion. The experimental results indicated that the buckling strain of FRP tube with in-filled concrete was higher than that of a hollow FRP tube (Chapters 3 and 4). A factor of k_2 is introduced to account for this beneficial effect that delays the onset of buckling in an FRP tube. Therefore the estimated buckling strain of FRP tube is:

$$\varepsilon_{cb} = \frac{k_2}{k_1} \frac{f_{ul,c}}{E_l} \tag{6.8}$$

If the axial strain of specimen reaches \mathcal{E}_{cb} , the buckling of FRP tube occurs.

As may be expected, a correlation exists between the confining stiffness (ρ_K) and k_2 (Fig. 6.8), in which k_2 is back-calculated from the experiment results. The confining stiffness is define by Eq. (6.9), which is adopted from Teng et al. (2009). Eq. (6.10) is proposed to estimate the beneficial effect of concrete core on tube buckling by a regression analysis of experimental data (Fig. 6.8). As shown in Eq. (6.10) and Fig. 6.8, k_2 is equal to 1 if no concrete exists and an upper limit for k_2 is set to 2 based on the available experimental data.

$$\rho_{K} = \frac{2E_{h}t_{o}}{(f_{co}^{'} / \varepsilon_{co})D_{i}}$$
(6.9)

$$k_{2} = \begin{cases} 4.12\rho_{K} + 1, & \text{if } \rho_{K} \le 0.226\\ 2, & \text{if } \rho_{K} > 0.226 \end{cases}$$
(6.10)

The performance of the above proposed buckling failure criterion in predicting FRP tube buckling is shown in Fig. 6.9. It is worthwhile to mention that the first occurrence of FRP tube buckling during an experiment is judged from its load-axial strain curve and load-hoop strain curve. FRP tube buckling normally caused a sudden drop of applied load and a dramatic change of strain gauge readings. Fig. 6.9 indicates that the prediction of tube buckling shows acceptable accuracy given the many complicating factors, including the complex lamination structure, biaxial strass state, interaction between tube and core concrete, and errors introduced in the determination of compressive properties of FRP tube.



Fig. 6.8. Determination of the relationship between k_2 and ρ_K



Fig. 6.9. Axial strain at FRP tube buckling: experiments versus predictions

6.3.3.2 Rupture of FRP tube

The ultimate condition of concrete-filled FRP tubes is reached when the tube is ruptured by hoop tension. The load corresponding to the ultimate condition the FRP tube is generally the maximum load a specimen can sustain. A descending type of load-axial strain curve is unlikely to appear due to the relatively large thickness of a filament-wound FRP tube in comparison to the small thickness a FRP wrap can have. Theoretically speaking, the hoop strength of FRP tube can be reduced due to the occurrence of tube buckling under longitudinal compression. However, the experimental results (Chapters 3 and 4) indicate that the ratio of rupture strain of concrete-filled FRP tube to the ultimate hoop strain obtained from the split-disk material property tests ranges from 0.91 to 1.26, with the average being 1.06 and COV (coefficient of variation) being 0.13. That is, the hoop rupture strain in a SWSSC-filled FRP tube can even be higher than the rupture strain from a split-disk test. This phenomenon can be attributed to the underestimation of the material hoop rupture strain by a split-disk test due to local bending at the gaps of such a test; for FRP wraps, their hoop rupture strain is commonly known to be significantly lower than that from a flat coupon test (Lam and Teng 2004) Therefore, the rupture of FRP tube due to hoop tension is deemed to occur when its hoop stress reaches the ultimate hoop strength (f_{ub}) from a split-disk test.

6.3.4 Load carried by FRP tube

A major difference between an FRP tube and an FRP wrap is that the tube posseses substantial strength and stiffness in the longitudinal direction. Fig. 6.10 summarizes the loads carried by the FRP tube and the core concrete respectively at the first occurrence of buckling and the contribution percentage of FRP tube (the top bar of the bar in Fig. 6.10). It is assumed that the maximum load carried by an FRP tube is equal to that carried by a corresponding hollow short tube in a material properties test. As shown in Fig. 6.10, the FRP tube shares a considerable part of the applied load, and its contribution should be accounted for in the theoretical model. The contribution of FRP tube to the load-carrying capacity increases with a decrease in the diameter-to-thickness ratio (D_0/t_0) or an increase in the longitudinal compressive strength of FRP ($f_{ul,c}$).

A simplified model is proposed to estimate the load carried by FRP tube as shown in Fig. 6.11 and Eqs. (6.11-12). It is assumed that the maximum load the FRP tube can carry is equal to the strength of a corresponding hollow short tube (N_{fu}). As discussed in Section 3.3.1, the buckling strain of an FRP tube filled with concrete is higher than that of a hollow short tube due to the beneficial effect of in-filled concrete. This assumption is thus generally conservative. Upon the buckling of FRP tube, a certain percentage of the load carried by the FRP tube is released as indicated by the load drop seen in the experimental load-axial strain curves (Fig. 6.2a). The maximum load drop observed for the test

specimens is about 40% of N_{fu} , and the corresponding reduction factor η is conservatively set to be 0.5 in the present study for simplicity. After the first occurrence of FRP tube buckling, it is assumed that the residual strength of FRP tube decreases linearly to zero until tube rupture. It is known that in an analysis-oriented stress-strain model for FRP-confined concrete, the ultimate axial strain (ε_{cu}) is only attained when the ultimate condition of the FRP tube is reached. Therefore, the hoop strain of FRP tube is adopted here to estimate the residual strength of FRP tube after the first occurrence of tube buckling (Eq. (6.13)).

$$N_{f} = \begin{cases} \min\{A_{f}\sigma_{l}, N_{fu}\}, & \text{if } \varepsilon_{c} \le \varepsilon_{cb} \\ (1-\eta)N_{fu}k_{3}, & \text{if } \varepsilon_{c} > \varepsilon_{cb} \end{cases}$$
(6.11)

$$N_{fu} = A_f f_{ul,c} \tag{6.12}$$

$$k_3 = 1 - \frac{\varepsilon_h - \varepsilon_{hb}}{\varepsilon_{hu} - \varepsilon_{hb}} \tag{6.13}$$

where ε_h is the hoop strain of FRP tube, ε_{hb} is the hoop strain of FRP tube at tube buckling, and ε_{hu} is the ultimate hoop strain of FRP tube.



Fig. 6.10. Percentage contribution of FRP tube to the load-carrying capacity of concrete-filled FRP tube



Fig. 6.11. Simplified load-axial strain model for FRP tube

6.3.5 Load-axial strain model

6.3.5.1 Stress-strain curve of confined concrete

A study (Chapter 2) on unconfined SWSSC has demonstrated that its mechanical behaviour is similar to that of normal concrete (denoted as "freshwater river sand concrete" in Chapter 2). Another study (Chen et al. 2017b) has concluded that the behaviour of FRP-confined SWSSC is similar to that of FRP-confined normal concrete. In the present study, it is assumed that the stress-strain behaviour of SWSSC is similar to that of normal concrete. In most of the existing analysis-oriented stress-strain models for FRP-confined concrete, the axial stress-strain relationship originally proposed by Popovics (1973) for actively confined concrete, which was used by Mander et al. (1988) for steel-confined concrete, was adopted as part of the active confinement base model. Popovics's model is rewritten as:

$$\sigma_c = \frac{f_{cc}^*(\varepsilon_c / \varepsilon_{cc}^*)r}{r - 1 + (\varepsilon_c / \varepsilon_{cc}^*)^r}$$
(6.14)

where r accounts for the brittleness of concrete and was defined by Carreira and Chu (1985) as:

$$r = \frac{E_c}{E_c - f_{cc}^{**} / \varepsilon_{cc}^{*}}$$
(6.15)

where E_c is the initial elastic modulus of concrete, which is taken as $4730\sqrt{f_{co}}$ (with f_{co} in MPa) in according with ACI 318-11 (2011).

Two approaches have been widely adopted by researchers to determine the peak stress (f_{cc}^*) of concrete at a given confining pressure (f_l). One was proposed by Mander et al. (1988) based on the "five-parameter" multiaxial failure surface described by William and Warnke (1975), which has been by Mirmiran and Shahawy (1997), Spoelstra and Monti (1999), Fam and Rizkalla (2001), Chun and Park (2002) and Aire et al. (2010). The other approach is a simple equation of the following form originally proposed by Richart et al. (1928):

$$f_{cc}^{*} = f_{co}^{'} + kf_l \tag{6.16}$$

Different values of k have been proposed by different researchers (e.g. Marques et al. 2004, Albanesi et al. 2007, and Teng et al. 2007). In the present study, Eq. (6.16) is adopted to determine the peak stress and k is set to 3.5 as suggested by Teng et al. (2007).

An expression originally proposed by Richart et al. (1928) has been most widely used to predict the axial strain (ε_{cc}^*) of confined concrete at peak stress. A modified version of Richart et al.'s equation was proposed by Jiang and Teng (2007) to predict ε_{cc}^* :

$$\frac{\varepsilon_{cc}^{*}}{\varepsilon_{co}} = 1 + 17.5 (\frac{f_{l}}{f_{co}})^{1.2}$$
(6.17)

Eq. (6.17) is also adopted in the present study to predict the axial strain at peak stress.

3.5.2 Numerical procedure

The proposed model for predicting the axial load-axial strain relationship needs to be implemented for two stages, separated by FRP tube buckling (Fig. 6.12). Before buckling, the biaxial behaviour of the FRP tube in resisting the applied load is accounted for. After buckling, the FRP tube is approximated an FRP wrap with a zero Poisson's ratio but the residual axial strength of the FRP tube is considered as discussed in Section 3.4. As shown in Fig. 6.12, a straight line (AB, ab) is adopted to connect the two parts.



Fig. 6.12. Conceptual illustration of proposed load-axial strain model

An iterative process is needed to develop the load-axial strain curve as summarized below. The hoop strain in the FRP tube can be determined for a given axial strain from the dilation model. The stresses in the FRP tube can then be calculated from the constitutive law (Eq. (6.5)) and the confining pressure can then be determined. The stress-strain relationship of actively confined concrete at this confining pressure can be predicted, and the axial stress in the concrete at the given axial strain can be found from the stress-strain relationship (Eqs. (6.14-17)). The loads carried by the FRP tube and the core concrete can then be determined respectively, and the sum of them is the load carried by the column:

$$N = \sigma_l A_c + N_f \tag{6.18}$$

The above steps are repeated to generate the entire load-axial strain curve until the rupture of the FRP tube. Fig. 6.13 summarizes the numerical procedure to generate the load-axial strain curve of concrete-filled FRP tube under axial compression. It should be mentioned that in the dilation model (Eq. (6.2)), the axial strain is expressed as a function of lateral strain (= hoop strain). For convenience in computation, a hoop strain increment (and hence the current hoop strain) can be first given and then the axial strain can be determined by the dilation model without iterations, as illustrated in Fig. 6.13. The hoop strain increment ($\Delta \varepsilon_h^{(i)}$) was adopted as a consistent value in the present study, while it also could change values during the iteration to increase the computation efficiency.



Fig. 6.13. Flowchart of numerical procedure

6.4 Verification of proposed theoretical model

6.4.1 Load-axial strain response

6.4.1.1 Authors' experiments

Comparisons between the predicted load-axial strain curves and the experimental curves from Chapters 3 and 4 are shown in Fig. 6.14. The details of the specimens are listed in Table 6.1. Generally, the predictions are in close agreement with the experimental data. For specimens with a diameter of 50 mm (i.e. C50-C and B50-C), the predicted curves provide substantially lower values than experimental curves. This can probably be attributed to two reasons: the predicted buckling strain is lower than the experimental value, and the contribution of FRP tube to the load-carrying capacity is significantly underestimated due to the higher fraction of tube area to the total cross-section area. It is noted that these two specimens are relatively small in size (50mm in diameter and 150mm in height), which may also affect the accuracy of prediction (i.e., size effect). It is worthwhile to note that these two specimens are of a small size that is unlikely to be used in engineering practice. A small overestimation is seen for the concrete-filled GFRP tubes (Fig. 6.14b), especially at the vicinity of tube buckling. The predicted load-axial strain curves for concrete-filled CFRP and BFRP tubes are in close agreement with the experimental curves (Figures 6.14c and d). However, the predicted ultimate capacity and ultimate axial strain are somewhat lower than the corresponding experimental data for concrete-filled CFRP tubes (i.e. C114-C and C165-C). It was found that the tube rupture strain of these specimens is about 25% higher than the rupture strain obtained from the split-disk test, which can explain the under-estimation of the ultimate condition. Fig. 6.14 indicates that the accuracy of the proposed model greatly relies on its accuracy in predicting the effect of buckling on the FRP tube.



Fig. 6.14. Comparison of load-axial strain response between predictions and authors' experiments

6.4.1.2 Other researchers' experiments

While a large number of studies have focused on the behaviour of concrete-filled FRP wraps, there have been limited experimental results for concrete-filled FRP tube. Table 6.3 summarizes some of the available axial compressive experiments (Fam and Rizkalla (2001), El Chabib et al. (2005) and Mohamed and Masmoudi (2010)) on concrete-filled FRP tubes having considerable longitudinal strength and stiffness. The hoop strength-to-longitudinal strength ratio ($f_{uh}/f_{ul,c}$) ranges from 2.45 to 6.48 and the unconfined concrete strength is less than 60 MPa. A close agreement between the predictions and the experimental curves (Fig. 6.15) further demonstrates the capability of the proposed model in predicting the load-axial strain response of concrete-filled FRP tubes under axial compression. It should be pointed out that the experimental ultimate axial strain (around 0.01) reported in Fam and Rizkalla's (2001) is much lower than those (around 0.04 to 0.05) reported in El Chabib et al. (2005) and Mohamed and Masmoudi (2010) and in Fig. 6.14 of the current tests. Furthermore, the buckling of FRP tubes occurred at an axial strain around 0.01 in all the tests reported in Chabib et al. (2005) and Mohamed and Masmoudi (2010) and Fig. 6.14.



Fig. 6.15. Comparison of load-axial strain response between predictions and other researchers' experiments

Data course	Specimen	Stacking sequence	$D_o ({ m mm})$	t_o (mm)	f_{co}' (MPa)	Longitudinal direction				Hoop direction		
Data source						$f_{ul,t}$ (MPa)	$f_{ul,c}$ (MPa)	E_l (GPa)	v_l	f_{uh} (MPa)	E_h (GPa)	$v_h{}^b$
	Stub 1	[8/-86/-86/8/-86/8/-85/8/-86]	168.0	2.56 ^a	58.0	282.9	224.1	19.8	0.07	548.0	33.4	0.11
Form and Digitalla 2001	Stub 2	[8/-86/-86/8/-86/8/-85/8/-86]	168.0	2.56 ^a	58.0	282.9	224.1	19.8	0.07	548.0	33.4	0.11
Fain and Kizkana 2001	Stub 8	[-88/-88/4/-88/-88/4/-88]	219.0	2.21ª	58.0	201.3	182.6	19.8	0.06	548.0	33.4	0.09
	Stub 11	[-87/3/-87/3/-87/3/-87]	100.0	3.08	37.0	444.0	115.0	29.0	0.10 ^b	398.0	23.0	0.08
El Chabib et al. 2005	NC	[±55]	162.0	6.00	39.5	N/A	60.0	8.5	0.39	193.5	10.5	0.48
El Chablo et al. 2003	SCC ^c	[±55]	162.0	6.00	43.8	N/A	60.0	8.5	0.39	193.5	10.5	0.48
Mahamad & Maamandi 2010	C30	$[\pm 65_3/\pm 45/\pm 65_3]$	164.8	6.40	30.0	60.2	N/A	9.3	0.29 ^b	390.0	23.6	0.74
wonamed α wasmoudi 2010	C45	$[\pm 65_3/\pm 45/\pm 65_3]$	164.8	6.40	45.0	60.2	N/A	9.3	0.29 ^b	390.0	23.6	0.74

Table 6.3. Specimen details of other researchers' experiments on concrete-filled FRP tubes

^a: Excluding the thickness of liner; ^b: Calculated by classical laminate theory; ^c: Self-compacting concrete

6.4.2 FRP tube buckling

The predicted FRP tube buckling loads and corresponding experimental results from Chapters 3 and 4 are compared in Fig. 6.16. The details of the specimens are given in Section 2. Even though there are some scatters in the prediction of buckling strain as shown in Fig. 6.9, the predictions for the buckling loads of FRP tubes are accurate (Fig. 6.16). It should be mentioned that the buckling of concrete-filled FRP tubes with a substantial longitudinal stiffness does not appear to have been examined in previous studies. However, FRP tube buckling was clearly observed in the authors' experiments (Chapters 3 and 4), which occurred with a loud sound and obvious change of tube appearance.



Fig. 6.16. FRP tube buckling loads: comparison between predictions and experiments

6.4.3 Ultimate condition

The ultimate load and the ultimate axial strain are defined as the load and the axial strain corresponding to the occurrence of FRP tube rupture. In Fig. 6.17, the predictions of the proposed model are compared with the experimental results from existing studies (Chapters 3 and 4; Fam and Rizkalla 2001; El Chabib et al. 2005; Mohamed and Masmoudi 2010). Close agreement can be seen in Fig. 6.17, and the predictions for ultimate loads show higher accuracy than predictions for ultimate axial strains, which is commonly seen in predictions for FRP-confined concrete columns (e.g. Teng et al. 2007; Binici 2005). In the proposed model, both the longitudinal strength and stiffness are ignored when the specimen reaches the ultimate condition. The method adopted to predict the ultimate load and the ultimate strain is same as that in Teng et al.'s model (2007).



Fig. 6.17. Comparison of ultimate condition between predictions and experiments

6.5. Conclusions

This chapter has presented a theoretical model for the compressive behaviour of FRP tubular columns filled with seawater and sea sand concrete (SWSSC), in which the FRP tube possesses substantial stiffness and strength. The confined concrete in such tubes is depicted using an existing analysis-oriented stress-strain model in combination with a biaxial model for the FRP tube. The effect of local buckling of the FRP tube is given due attention. The following conclusions are made.

(1) Teng et al.'s (2007) dilation model (lateral-axial strain relationship) can be used to closely predict the behaviour of SWSSC under FRP tube confinement provided the biaxial behaviour of the FRP tube is properly accounted for. The predictions of this adapted dilation model are in close agreement with available experimental results.

(2) A maximum strain criterion with appropriate improvements has been proposed in this study to predict the first occurrence of local buckling in FRP tubes, which is a likely phenomenon of concrete-filled FRP tubes with substantial longitudinal stiffness. FRP tube buckling is governed by both the material properties of the FRP tube and the interaction between the tube and encased concrete.

(3) The significant contribution of FRP tube to the load-carrying capacity should not be ignored, and a simplified load-axial strain model has been proposed to determine the load carried by the FRP tube throughout the whole loading process.

(4) A two-stage theoretical model has been proposed for concrete-filled FRP tubular stub columns under axial compression. For stage one (before tube buckling), the biaxial behaviour of the FRP tube, including the Poisson's effect, is considered in the model. For stage two (after tube buckling), the concrete-filled FRP tube is assumed to behave as a concrete-filled FRP wrap except that the residual

strength of the FRP tube is appropriately considered in the proposed model. The predictions from the proposed model are in close agreement with available experimental results.

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References

ACI 318-11 (2011), Building code requirements for structural concrete and commentary. American Concrete Institute, Farmington Hills, MI.

Aire, C., Gettu, R., Casas, J., Marques, S. and Marques, D. (2010), Concrete laterally confined with fibre-reinforced polymers (FRP): experimental study and theoretical model, Materiales de Construcción, 60, 19-31.

Albanesi, T., Nuti, C. and Vanzi, I. (2007), Closed form constitutive relationship for concrete filled FRP tubes under compression, Construction and Building Materials, 21, 409-27.

ASTM D3039-D3039M-14 (2014), Standard test method for tensile properties of polymer matrix composite materials, American Society for Testing and Materials, West Conshohocken, PA.

Binici, B. (2005), An analytical model for stress–strain behavior of confined concrete, Engineering Structures, 27, 1040-51.

Carreira, D.J. and Chu, K.-H. (1985), Stress-strain relationship for plain concrete in compression, Journal Proceedings, 797-804.

Chen, G.M., He, Z.B., Jiang, T., Chen, J.F. and Teng, J.G. (2017a), Axial compression tests on FRPconfined seawater/sea-sand concrete, 6th Asia-Pacific Conference on FRP in Structures.

Chen, G.M., He, Z.B., Jiang, T., Chen, J.F. and Teng, J.G. (2017b), Effects of seawater and sea-sand on the behaviour of FRP-confined concrete, 6th Asia-Pacific Conference on FRP in Structures.

Chun, S. and Park, H. (2002), Load carrying capacity and ductility of RC columns confined by carbon fiber reinforced polymer, 3rd International Conference on Composites in Infrastructure, University of Arizona, San Francisco.

Daniel, I.M. and Ishai, O. (2006), Engineering mechanics of composite materials, 2rd ed, Oxford University Press.

El Chabib, H., Nehdi, M. and El Naggar, M.-H. (2005), Behavior of SCC confined in short GFRP tubes, Cement and Concrete Composites, 27, 55-64.

Fam, A., Greene, R. and Rizkalla. S. (2003), Field applications of concrete-filled FRP tubes for marine piles, ACI Special Publication, 215, 161-80.

Fam, A. and Rizkalla, S. (2001a), Behavior of axially loaded concrete-filled circular FRP tubes, ACI Structural Journal, 98, 280-9.

Fam, A. and Rizkalla, S. (2001b), Confinement model for axially loaded concrete confined by circular fiber-reinforced polymer tubes, ACI Structural Journal, 98, 451-61.

Harries, K.A. and Kharel, G. (2002), Behavior and modeling of concrete subject to variable confining pressure, ACI Materials Journal, 99, 180-9.

Jiang, T. and Teng, J.G. (2007), Analysis-oriented stress–strain models for FRP–confined concrete, Engineering Structures, 29, 2968-86.

Lam, L. and Teng, J.G. (2003), Design-oriented stress-strain model for FRP-confined concrete, Construction and Building Materials, 17, 471-89.

Lam, L. and Teng, J.G. (2004), Ultimate condition of fiber reinforced polymer-confined concrete, Journal of Composites for Construction, 8, 539-48.

Li, Y.L., Zhao, X.L., Raman Singh, R.K. and Al-Saadi, S. (2016a), Experimental study on seawater and sea sand concrete filled GFRP and stainless steel tubular stub columns, Thin-Walled Structures, 106, 390-406.

Li, Y.L., Zhao, X.L., Raman Singh, R.K. and Al-Saadi, S. (2016b), Tests on seawater and sea sand concrete-filled CFRP, BFRP and stainless steel tubular stub columns, Thin-Walled Structures, 108, 163-84.

Li, Y.L., Zhao, X.L., Singh Raman R.K. and Al-Saadi, S. (2018), Thermal and mechanical properties of alkali-activated slag paste, mortar and concrete utilising seawater and sea sand, Construction and Building Materials, 159, 704-24.

Lim, J.C. and Ozbakkaloglu, T. (2014), Lateral strain-to-axial strain relationship of confined concrete, Journal of Structural Engineering, 141:04014141.

Mander, J.B., Priestley, M.J. and Park, R. (1988), Theoretical stress-strain model for confined concrete, Journal of Structural Engineering, 114, 1804-26.

Marques, S.P.C., Marques, D., Lins da Silva, J. and Cavalcante, M.A.A. (2004), Model for analysis of short columns of concrete confined by fiber-reinforced polymer, Journal of Composites for Construction, 8, 332-40.

Mirmiran A and Shahawy M. (1997), Dilation characteristics of confined concrete, Mechanics of Cohesive - Frictional Materials, 2, 237-49.

Mirmiran, A. (2003), Stay-in-place FRP form for concrete columns, Advances in Structural Engineering, 6, 231-41.

Mohamed, H.M. and Masmoudi, R. (2010), Axial load capacity of concrete-filled FRP tube columns: experimental versus theoretical predictions, Journal of Composites for Construction, 14, 231-43.

Ozbakkaloglu, T., Lim, J.C. and Vincent, T. (2013), FRP-confined concrete in circular sections: review and assessment of stress–strain models, Engineering Structures, 49, 1068-88.

Ozbakkaloglu, T., Lim, J.C. (2013), Axial compressive behavior of FRP-confined concrete: Experimental test database and a new design-oriented model, Composites Part B: Engineering, 55, 607-34.

Ozbakkaloglu, T. and Xie, T. (2016), Geopolymer concrete-filled FRP tubes: behavior of circular and square columns under axial compression, Composites Part B: Engineering, 96, 215-30.

Popovics, S. (1973), A numerical approach to the complete stress-strain curve of concrete, Cement and Concrete Research, 3, 583-99.

Richart, F.E., Brandtzaeg, A. and Brown, R.L. (1928), A study of the failure of concrete under combined compressive stresses, University of Illinois at Urbana Champaign, College of Engineering. Engineering Experiment Station.

Sfer, D., Carol, I., Gettu, R. and Etse, G. (2002), Study of the behavior of concrete under triaxial compression, Journal of Engineering Mechanics, 128, 156-63.

Smith, S.S., Willam, K.J., Gerstle, K.H. and Sture, S. (1989), Concrete over the top--or, is there life after peak?, ACI Materials Journal, 86, 491-7.

Spoelstra, M.R. and Monti, G. (1999), FRP-confined concrete model, Journal of Composites for Construction, 3, 143-50.

Teng, J.G., Huang, Y.L., Lam, L. and Ye, L.P. (2007), Theoretical model for fiber-reinforced polymer-confined concrete, Journal of Composites for Construction, 11, 201-10.

Teng, J.G., Jiang, T., Lam, L. and Luo, Y.Z. (2009), Refinement of a design-oriented stress–strain model for FRP-confined concrete, Journal of Composites for Construction, 13, 269-78.

Teng, J.G. and Lam, L. (2004), Behavior and modeling of fiber reinforced polymer-confined concrete, Journal of Structural Engineering, 130, 1713-23.

Teng, J.G., Yu, T., Dai, J.G. and Chen, G.M. (2011), FRP composites in new construction: current status and opportunities, in: Proceedings of 7th National Conference on FRP Composites in Infrastructure (Supplementary Issue of Industrial Construction), keynote presentation.

Teng, J.G. (2014), Performance enhancement of structures through the use of fibre-reinforced polymer (FRP) composites, in: Proceedings of 23rd Australasian Conference on the Mechanics of Structures and Materials (ACMSM23), Lismore, Australia, keynote presentation.

Wang, Z., Zhao, X.-L., Xian, G., Wu, G., Singh Raman, R.K., Al-Saadi, S. et al. (2017a), Long-term durability of basalt-and glass-fibre reinforced polymer (BFRP/GFRP) bars in seawater and sea sand concrete environment, Construction and Building Materials, 139, 467-89.

Wang, Z., Zhao, X.-L., Xian, G., Wu, G., Singh Raman, RK, and Al-Saadi, S. (2017b), Durability study on interlaminar shear behaviour of basalt-, glass- and carbon-fibre reinforced polymer (B/G/CFRP) bars in sea water sea sand concrete environment, Construction and Building Materials, 156, 985-1004.

William, K.J. and Warnke, E.P. (1975), Constitutive model for the triaxial behavior of concrete, International Association for Bridge and Structural Engineering Proceedings, 19, 1-30.

Wu, G., Lü, Z. and Wu, Z. (2006), Strength and ductility of concrete cylinders confined with FRP composites, Construction and Building Materials, 20, 134-48.

Xie, P., Lin, G., Teng, J,G. and Yu, T. (2017), Modelling of concrete-filled FRP tubes considering nonlinear biaxial tube behavior, 6th Asia-Pacific Conference on FRP in Structures.

Yu, T., Zhang, B. and Teng, J.G. (2015), Unified cyclic stress-strain model for normal and high strength concrete confined with FRP, Engineering Structures, 102, 189-201.

Zhang, B., Yu, T. and Teng, J.G. (2014), Behavior of concrete-filled FRP tubes under cyclic axial compression, Journal of Composites for Construction, 19, 04014060.

Chapter 7

Analysis-oriented load-strain model for concrete-filled doubleskin circular FRP tubes under axial compression
Abstract

Concrete-filled double-skin FRP tubes (CFDST) are increasingly attracting researchers' interests due to the advantages of their reduced self-weight and higher bending stiffness than fully filled tubes. However, the structural behaviour of CFDST, especially the non-uniform confinement in annular concrete, has not ever been well addressed. This chapter presents an analytic study on axial compressed circular stub CFDST with FRP wrap/tube as outer tube and steel/FRP as inner tube. Based on existing studies on actively confined concrete, a constitutive model for non-uniformly FRP-confined concrete is developed in the present study. The dilation model for concrete fully filled FRP tubes is modified to account for the effects of void ratio so that the hoop-axial strain curve of CFDST could be reasonably predicted. Behaviours of steel and FRP tubes in CFDST are investigated and proper stress-strain models are proposed to estimate the loads shared by tubes. The stress state in annular concrete is theoretically studied by dividing the cross-section into multiple circular layers. Finally, an analysis-oriented load-strain model, which accounts for the non-uniform confinement, effects of void ratio, buckling of FRP tube, and strain hardening of stainless steel, is proposed for CFDST. As validated by the experimental data from a wide range of literature, the proposed model shows good reasonability and high accuracy.

Keywords

Concrete-filled double-skin FRP tube; non-uniform confinement; dilation; load-strain model; axial compression

Nomenclature

a_0, a_1, a_2	constants for defining the tensile meridian in Willam-Warnke model
$A_{ m c}$	cross-sectional area of concrete
A_{i}	cross-sectional area of inner FRP tube
В	width of cubic specimen
b_0, b_1, b_2	constants for defining the compressive meridian in Willam-Warnke model
D	diameter of cylindrical specimen
D_{i}	outer diameter of inner tube
$D_{ m o}$	outer diameter of outer tube
$E_{\rm c}$	Young's modulus of unconfined concrete
$E_{ m h}$	Young's modulus of FRP tube in hoop direction
E_l	Young's modulus of FRP tube in longitudinal direction
$E_{\rm s}$	initial Young's modulus of stainless steel
$f_{0.2}$	0.2% proof stress of stainless steel
$f_{\rm c}$ '	unconfined concrete strength
f_{cc}	confined concrete strength
f_l	confining stress

$f_{\rm uh}$	hoop tensile strength of FRP wrap/tube
ful	longitudinal compressive strength of FRP tube
Ja	second invariant of deviatoric stresses
k_1, k_2, k_3	factors
$k_{\rm ef}$	effective factor for rupture strain of FRP wrap
N	number of divided layers in annular cross-section
N _{fi}	load carried by inner FRP tube
N _{fui}	ultimate capacity of inner FRP tube
r	parameter accounting for concrete brittleness
<i>r</i> i	radius of the i th divided laver
Ri	inner radius of annular concrete
R_{0}	outer radius of annular concrete
t;	thickness of inner tube
t _o	thickness of outer tube
50 50	nominal vielding strain of stainless steel
с.	axial strain
E-h	axial strain at tube buckling (predicted)
Сс) Е	axial strain at confined concrete strength $f_{\rm ex}$?
600 8	axial strain at unconfined concrete strength f_{cc}^{*}
600 E	ultimate axial strain
Ecu CEET	ultimate axial strain of concrete fully-filled FRP tube (CFFT)
Ecu CEDST	ultimate axial strain of concrete filled double-skin FRP tube (CFDST)
Ef	ultimate strain of FRP wrap (by flat coupon test)
Eh .	hoop strain
Ehb	hoop strain at FRP tube buckling
Ehu	Hoop rupture strain of FRP wrap/tube
ε ₁	longitudinal strain in FRP tube
Elat	lateral strain of concrete
\mathcal{E}_{LB}	ultimate strain of stainless steel circular hollow section
η	load drop factor
θ	Lode angle
$\lambda_{\rm c}$	cross-section slenderness of stainless steel circular hollow section
$v_{\rm h}$	Poisson's ratio of FRP tube (hoop tension)
v_l	Poisson's ratio of FRP tube (longitudinal compression)
$v_{\rm s}$	Poisson's ratio of stainless steel
ρ	variable for defining the deviatoric part of a stress tensor
$ ho_{ m K}$	confinement stiffness
$\sigma_1, \sigma_2, \sigma_3$	principle stresses
$\sigma_{ m c}$	axial stress in concrete
$\sigma_{ m h}$	hoop stress in FRP wrap/tube
σ_l	longitudinal stress in FRP tube
$\sigma_{ m m}$	mean stress
$\sigma_{ m r}$	radial stress
$\sigma_{\rm z}$	stress in z-axis
$\sigma_{ heta}$	circumferential stress
φ	void ratio

7.1 Introduction

Fiber reinforced polymer (FRP) has been increasingly used in civil engineering due to its features of high strength, light weight and corrosion resistance (Bank 2006; Hollaway and Teng 2008; Park and Choi 2013; Zhao 2013; Wang et al. 2014). The strength and ductility of concrete could be significantly enhanced when it is confined by FRP composites (e.g., FRP wrap in retrofitting and FRP tube in new construction). A new structural member form (called "hybrid FRP-concrete-steel tube" or "concretefilled double-skin FRP tube"), which consists of an outer FRP wrap, an inner steel tube and concrete in-filled between them, was recently proposed by Teng et al. (2007a). This form of construction is similar to the conventional concrete-filled double skin tubes (CFDST) (e.g. Zhao and Han 2006, Han et al. 2009, Zhao et al. 2002, 2010) except that FRP tubes are used to replace carbon steel tubes. As compared to concrete fully filled tubes (CFFTs), concrete-filled double-skin tubes (CFDST) have the advantages of reduced self-weight and higher bending stiffness. In order to extend the application of CFDST to new structures in a marine environment, the authors have explored the possibilities of replacing FRP wrap by FRP tube, inner carbon steel tube by stainless steel or FRP tube, and ordinary Portland cement concrete by seawater and sea sand concrete (SWSSC), with promising results (Li et al. 2016a, b, 2018a, b, c; Wang et al. 2017; Wang et al. 2018). Therefore, an accurate theoretical model is needed to assess the structural behaviour of CFDST for their further applications.

Extensive research has been conducted on concrete fully filled FRP tubes (CFFTs), whereas the studies on concrete-filled double-skin FRP tubes (CFDST) are limited. The studies (Wong et al. 2008; Yu et al. 2009; Ozbakkaloglu and Fanggi 2013; Fanggi and Ozbakkaloglu 2015) on CFDST (FRP wrap as outer tube and carbon steel as inner tube) under axial compression indicated that: (a) the inward expansion of sandwich concrete could be restrained by inner steel tube; (b) CFDST was effectively confined by FRP wrap and behaved similarly to the fully filled wrap/tube; (c) with the increase of void ratio, the ultimate strain of a CFDST increased considerably whilst only slight strength increase was observed. Zhang et al. (2017) investigated the compressive behaviour of CFDST with carbon steel inner tube and filament-wound FRP outer tube, in which the fiber orientation was $\pm 80^{\circ}$ and the axial-to-hoop strength ratio was 0.14. Two CFDST (FRP as both outer and inner tubes) were tested by Fam and Rizkalla (2001a) and the axial-to-hoop strength ratios were 0.41 and 0.92 for outer and inner tubes, respectively. The authors of the present article conducted a series of compressive tests on CFDST (FRP tube as outer tube and FRP/stainless steel as inner tube) and CFFTs (Chapters 3 and 4). It was found that the buckling of FRP tube could lead to a sudden load drop but the specimen could still increasingly sustain the applied load. Concrete was not

effectively confined in some CFDST with FRP as inner tube since the inner tube was seriously damaged by progressive buckling.

It is well known that the strength of concrete could be greatly enhanced due to the confinement effect provided by encasing FRP wrap/tube. The confinement provided by FRP is called "passive confinement" because the confining pressure increases with the increase of concrete lateral expansion, which is different from the "active confinement" where the confining pressure keeps constant. Existing stress-strain models are generally classified into two groups (Teng and Lam 2004): design-oriented model, which is in closed-form expressions, and analysis-oriented model, which needs an incremental iterative numerical procedure. In an analysis-oriented model, the interaction between the core concrete and the confining FRP is considered explicitly. The theory for actively confined concrete is often the base for developing analysis-oriented models (Mander et al. 1988).

Many stress-strain models for concrete fully-filled FRP wraps have been proposed in recent decades (Ozbakkaloglu et al. 2013), but the models for concrete-filled double-skin wraps/tubes are rather scarce. Fam and Rizkalla (2001b) proposed an analysis-oriented model for concrete both fully and partially filled FRP tubes (with central holes but without inner tube). Based on Gerstle's octahedral theory (Gerstle 1981a, b), Becque et al. (2003) proposed an analysis-oriented model for concrete-filled double-skin tubes (FRP as both inner and outer tubes), in which the concrete is assumed to be in a state of uniform octahedral stress. With respect to concrete-filled double-skin FRP wraps (carbon steel as inner tube), Yu et al. (2010) and Ozbakkaloglu et al. (2016) proposed design-oriented stress-strain models, which were the modified forms of the models for concrete fully filled FRP wraps by assuming the annular concrete was uniformly confined.

After reviewing the literature, it is found that none of the existing models have accounted for the nonuniform confinement in an annular concrete, which probably affected the prediction accuracy. The structural behaviours (e.g., load-carrying capacity, buckling) of inner stainless steel tube or FRP tube in a CFDST have not yet been well addressed in previous research. This chapter aims to fill the above mentioned knowledge gaps by proposing an analysis-oriented load (or stress)-axial strain model. A constitutive model for non-uniformly confined concrete was firstly developed in this study. The dilation properties and structural behaviour of FRP and stainless steel tubes were then presented. Finally, an analysis-oriented load-axial strain model was proposed and validated by the experimental data of the authors and other researchers.

7.2 Constitutive model for non-uniformly confined concrete

7.2.1 Failure criterion for concrete

Based on the theory of elasticity (Chen and Saleeb 1994), a stress tensor, which represents a stress state, could be geometrically decomposed into two parts: hydrostatic and deviatoric parts. The stress state could be represented by three parameters: σ_m , ρ , and θ (Fig. 7.1).

$$f_{cc}' = f_c' + 3.5 f_l \tag{7.1}$$

$$\rho = \sqrt{2J_2} \tag{7.2}$$

$$\cos\theta = \frac{2\sigma_1 - \sigma_2 - \sigma_3}{2\sqrt{3J_2}}$$
(7.3)

$$J_2 = \frac{2}{3} \left[\left(\frac{\sigma_1 - \sigma_2}{2} \right)^2 + \left(\frac{\sigma_1 - \sigma_3}{2} \right)^2 + \left(\frac{\sigma_2 - \sigma_3}{2} \right)^2 \right]$$
(7.4)

where σ_1 , σ_2 , and σ_3 are principle stresses (tension as positive), θ is Lode angle, and J_2 is the second invariant of deviatoric stresses. For a certain failure criteria, the relationship between σ_m and ρ could be illustrated in a meridian plane whereas the relationship of ρ and θ could be presented in a deviatoric plane (Fig. 7.1).



There are two extreme meridian planes which are compressive meridian ($\theta = 60^{\circ}$) and tensile meridian ($\theta = 0^{\circ}$). If a hydrostatic stress state ($\sigma_1 = \sigma_2 = \sigma_3$) is superposed by a compressive stress in one direction (Eq. (7.5)), it yields $\theta = 60^{\circ}$ and the meridian is called compressive meridian. Most of the existing experimental data falls on this meridian, including uniaxial compressive strength and equal biaxial tensile strength. On the other hand, if a hydrostatic stress state is superposed by a tensile stress in one direction (Eq. (7.6)), it is called tensile meridian ($\theta = 0^{\circ}$). Data on this meridian includes the uniaxial tensile strength and equal biaxial compressive strength as its special cases (Chen and Saleeb 1994).

$$\sigma_r = \sigma_1 = \sigma_2 > \sigma_z = \sigma_3 \tag{5}$$

$$\sigma_r = \sigma_2 = \sigma_3 < \sigma_z = \sigma_1 \tag{6}$$

where σ_r is radial stress, σ_z is axial (i.e., z-axis) stress, and tensile stress is set as positive.

Over the last century, many failure models have been proposed for concrete. Most of the models were defined in stress spaces by a number of material constants varying from one to five independent parameters. Two typical failure models, namely "Drucker-Prager two-parameter model" and "Willam-Warnke five-parameter model", are illustrated in Fig. 7.1. A linear relationship between ρ and σ_m is assumed in Drucker-Prager model and the ratio of ρ_t to ρ_c is equal to one, which yields a coincidence of the compressive and tensile meridians. Five parameters are implemented in Willam-Warnke model, by which the compressive and tensile meridians are depicted independently. The relationship between ρ and σ_m is assumed to be nonlinear and the ratio of ρ_t to ρ_c (<1.0) varies with σ_m .

Concrete confined by circular FRP tubes is under uniform confinement and its strength falls on the compressive meridian ($\theta = 60^{\circ}$). The existing analysis-oriented models assume a linear relationship between confined concrete strength (f_{cc} ') and confining stress (f_i), which is similar to the Drucker-Prager model shown in Fig. 7.1. However, the radial (σ_r) and circumferential (σ_{θ}) stresses are unequal ($\theta \neq 60^{\circ}$) in the annular sectional concrete confined by FRP. Because of the aforementioned features of Willam-Warnke model, it is more suitable to predict the strength of non-uniformed confined concrete.

7.2.2 Willam-Warnke model for non-uniformly confined concrete

Willam-Warnke model with five parameters (Willam and Warnke 1975 as described in Chen and Saleeb 1994) is adopted in this study and its validity will be further proved. The compressive and tensile meridians are expressed in parabolic forms:

$$\sigma_m = a_0 + a_1 \rho_t + a_2 \rho_t^2 \tag{7.7}$$

$$\sigma_m = b_0 + b_1 \rho_c + b_2 \rho_c^2 \tag{7.8}$$

where "t" stands for tension and "c" represents compression. Based on the calibration of Chen and Saleeb (1994), the values of the constants are $a_0=0.1025$, $a_1=-0.8403$, $a_2=-0.0910$, $b_0=a_0$, $b_1=-0.4507$, and $b_2=-0.1018$. Parameter ρ can be obtained by:

$$\rho(\theta) = \frac{2\rho_c(\rho_c^2 - \rho_t^2)\cos\theta + \rho_c(2\rho_t - \rho_c)[4(\rho_c^2 - \rho_t^2)\cos^2\theta + 5\rho_t^2 - 4\rho_t\rho_c]^{1/2}}{4(\rho_c^2 - \rho_t^2)\cos^2\theta + (\rho_c - 2\rho_t)^2}$$
(7.9)

The existing experimental data of uniformly, actively confined concrete (i.e., "triaxial vessel"), equal biaxially compressed concrete and true triaxially compressed concrete was collected (as summarized in Table 7.1). Fig. 7.2 presents a comparison between the experimental data and the predictions by Willam-Warnke model in meridian planes, in which the experimental data was grouped by unconfined concrete strength. The widely used relationship (Eq. (7.10)) between FRP-confined concrete strength f_{cc} and confining stress f_l ($\theta = 60^\circ$) is also shown in Fig. 7.2 (denoted as "Teng et al. 2007" given in Teng et al. 2007b).

$$f_{cc}^{'} = f_{c}^{'} + 3.5f_{l} \tag{7.10}$$

Fig. 7.2 indicates the Willam-Warnke model with the parameters determined by Chen and Saleeb (1994) to be reasonable to estimate the uniformly confined concrete, which is the base for its further application to non-uniformly confined concrete. It is necessary to mention that σ_m/f_c ' of FRP confined concrete generally ranges from -0.5 to -2.5, which could be covered by the actively confined concrete data collected in Table 7.1.



Fig. 7.2. Comparison of experimental data to Willam-Warnke model predictions in meridian plane

The available experimental data of unequally, triaxially compressed concrete was also collected in Table 7.1 (0°< θ <60°). The experimental data and predictions by Willam-Warnke model is compared in the deviatoric plane (Fig. 7.3). As the predictions are a function of σ_m/f_c ', the experimental data was grouped by the values of σ_m/f_c ' and the predicted curves at a given σ_m/f_c ' are shown in Fig. 7.3 by the dashed lines. If the experimental data falls in the area between two correspondingly predicted curves, the prediction is regarded as accurate. It is found that the prediction becomes less accurate with increasing σ_m/f_c ', i.e. with a higher level of confining stress. However, the high level of confining (i.e., σ_m/f_c ' <-2.5) is unlikely to occur in FRP-confined concrete. Fig. 7.4 presents a direct comparison between experimental strengths of non-uniformly confined concrete and strengths predicted by Willam-Warnke model. As shown in Fig. 7.4, the Lode angle (θ), which is related to the ratio of

confining stresses in two directions (σ_1/σ_2), does not obviously affect the prediction accuracy. Generally speaking, Willam-Warnke model could precisely predict the strength of non-uniformly confined concrete.

Lode angle, θ	Data source	Confinement type	Number of data	D (mm)	<i>B</i> (mm)	fc' (MPa)	f_l / $f_{ m c}$ '
	Cordon and Gillespie (1963)	Triaxial vessel	53	152.4		12.2 - 51.0	0.05 - 1.13
	Imran and Pantazopoulou (1996)	Triaxial vessel	36	54.0		21.2 - 73.4	0.04 - 1.00
	Lan and Guo (1997)	True triaxial	24		70.7	24.0	0.30 - 0.90
	Li and Ansari (1999)	Triaxial vessel	14	101.0		47.2 - 107.3	0.18 - 0.93
0-600	Li and Ansari (2000)	Triaxial vessel	11 76.2			69.9, 103.5	0.07 - 0.99
<i>θ</i> =60°	Lim and Ozbakkaloglu (2015)	Triaxial vessel	31	63.0		51.6, 128	0.02 - 0.48
	Lu and Hsu (2007)	Triaxial vessel	10	100.0		67.0	0.05 - 0.84
	Mills and Zimmerman (1970)	True triaxial	15		57.2	23.0 - 36.1	0.04 - 0.38
	Sfer et al. (2002)	Triaxial vessel	10	152.0		32.8, 38.8	0.04 - 1.83
	Xie et al. (1995)	Triaxial vessel	26	55.5		60.2 - 119.0	0.01 - 0.50
	Lee et al. (2004)	Equal biaxial	2		200.0	30.3, 39.0	1.16, 1.17
0-0°	Lan and Guo (1997)	True triaxial	8		70.7	23.9 - 24.1	2.00 - 3.50
<i>θ</i> =0°	Mills and Zimmerman (1970)	True triaxial	11		57.2	23.0 - 36.1	1.42 - 3.14
	Wang et al. (1987)	True triaxial	1		101.6	10.7	3.58
0°< <i>θ</i> < 60°	Mills and Zimmerman (1970)	True triaxial	46		57.2	9.4, 10.7	N/A
	Wang et al. (1987)	True triaxial	26		101.6	23.0 - 36.1	N/A

Table 7.1. Summary of experimental data of actively confined concrete



Fig. 7.3. Comparison of experimental data of non-uniformed confined concrete to the predictions in deviatoric plane



Fig. 7.4. Comparison of experimental strength of non-uniformed confined concrete to the predictions by Willam-Warnke model

7.2.3 Ultimate strain of non-uniformly confined concrete

The ultimate strain of non-uniformly confined concrete has not yet been experimentally reported. Only the ultimate strain in axial direction of uniformly confined concrete was collected in this study. A linear relationship (Eq. 7.11) was assumed between normalized strength (f_{cc}'/f_{c}') and strain ($\varepsilon_{cc}/\varepsilon_{co}$) by Richart et al. (1928) and it was most widely adopted by other researchers.

$$\frac{\varepsilon_{cc}}{\varepsilon_{co}} = 5 \cdot \frac{f_{cc}}{f_c} - 4 \tag{7.11}$$

where ε_{co} is ultimate strain of unconfined concrete, which equals to $0.000937 \sqrt[4]{f_c}$ (f_c ' in MPa) as suggested by Popovics (1973). Jiang and Teng (2007) modified Eq. (7.11) by substituting Eq. (7.10) into Eq. (7.11) and assuming a slightly nonlinear relationship between $\varepsilon_{cc}/\varepsilon_{co}$ and f_l/f_c ' (Eq. (7.12)). Eq. (7.12) has been successfully used in predicting the stress-strain curve of FRP confined concrete.

$$\frac{\varepsilon_{cc}}{\varepsilon_{co}} = 1 + 17.5 (\frac{f_{l}}{f_{c}})^{1.2}$$
(7.12)

Fig. 7.5 shows the experimental results and the curves predicted by the models of Richart et al. (1928) and Jiang and Teng (2007). The predictions by Eqs. (7.11) and (7.12) are similar and both are in good agreement with experimental data.

Due to the lack of strain data of non-uniformly confined concrete (i.e., $0 < \sigma_1/\sigma_2 < 1$), the concrete under biaxial compression (i.e., $\sigma_1=0$ and $\sigma_2 > \sigma_3$) and concrete under uniform confining stresses (i.e., $\sigma_1=\sigma_2>\sigma_3$) could be regarded as the extreme cases for non-uniformly confined concrete. Fig. 7.6 summarizes the experimental data of concrete under biaxial compression. The biaxial compressive strength is much lower than the triaxial compressive strength. As shown in Figs. 7.5 and 7.6, for a given f_{cc}'/f_{c}' , ultimate strain of concrete under biaxial compression is lower than that of concrete under uniformly triaxial compression. Therefore, Eqs. (7.11) and (7.12) will provide over-estimation on the ultimate strain of non-uniformly confined concrete.



Fig. 7.5. Ultimate strain of uniformly confined concrete



Fig. 7.6. Ultimate strain of concrete under biaxial compression

In order to keep it consistent with existing analysis-oriented model, Eq. (7.12) is modified by introducing a reduction factor to account for the effect of non-uniform confinement. The ultimate strain of non-uniformly confined concrete could be predicted by:

$$\frac{\varepsilon_{cc}}{\varepsilon_{co}} = 1 + 17.5 (\frac{0.6f_l}{f_c})^{1.2}$$
(7.13)

where f_l is calculated by Eq. (7.10) as f_{cc} ' has been estimated by Willam-Warnke model. Therefore, f_l can be regarded as an equivalent confining stress. The relationships of $\varepsilon_{cc}/\varepsilon_{co}$ to f_{cc}'/f_c' predicted by Eqs. (7.12) and (7.13) are plotted in Fig. 7.6. It should be pointed out that Eq. (7.13) with the factor of 0.6 is almost an upper bound of the experimental data of biaxial compressed concrete.

7.2.4 Stress-strain relationship of non-uniformly confined concrete

After knowing the confined concrete strength (f_{cc} ') determined by Willam-Warnke model and corresponding ultimate strain (ε_{cc}) by Eq. (7.13), the stress-strain relationship can be determined by

the method proposed by Popovics (1973). Since Popovics's method was derived from uniaxial compression test and the effects of non-uniform confinement have been considered in predicting f_{cc} ' and ε_{cc} , this method was directly adopted for non-uniformly confined concrete without modification. The stress-strain (σ_c - ε_c) relationship of concrete is written as:

$$\sigma_{c} = \frac{f_{cc}^{'}(\varepsilon_{c} / \varepsilon_{cc})r}{r - 1 + (\varepsilon_{c} / \varepsilon_{cc})^{r}}$$
(7.14)

$$r = \frac{E_c}{E_c - f_{cc}^{\prime} / \varepsilon_{cc}}$$
(7.15)

where *r* accounts for the brittleness of concrete (Carreira and Chu 1985), E_c is the initial elastic modulus of concrete (in MPa), which is taken as $4730\sqrt{f_c}$ (with f_c ' in MPa) in according with ACI 318-11 (2011).

7.3 Dilation property

7.3.1 Existing dilation model for concrete fully filled FRP tubes

The dilation property of concrete fully filled FRP wrap has been well understood. Among the various lateral-axial strain (ε_{lat} - ε_c) models (e.g., Mirmiran and Shahaway 1997; Spoelstra and Monti 1999; Fam and Rizkalla 2001b; Teng et al. 2007b and Lim and Ozbakkaloglu 2014a), the model proposed by Teng et al. (2007b) shows superior performance after experimentally comparing to other models (Jiang and Teng 2007), which is expressed as:

$$\frac{\varepsilon_c}{\varepsilon_{co}} = 0.85\{[1+0.75(\frac{-\varepsilon_{lat}}{\varepsilon_{co}})]^{0.7} - \exp[-7 \cdot (\frac{-\varepsilon_{lat}}{\varepsilon_{co}})]\} \cdot (1+8\frac{f_l}{f_{co}'})$$
(7.16)

$$f_l = \frac{\sigma_h t_o}{D_o / 2} \tag{7.17}$$

$$\sigma_h = E_h \varepsilon_h \tag{7.18}$$

where ε_{lat} is the lateral strain, f_l is the confining stress, σ_h is the hoop stress in FRP wrap.

In comparison to FRP wrap, FRP tube offers strength and stiffness in both axial and hoop directions. Chapter 6 proposed a hoop-axial strain (ε_h - ε_c) model for concrete-filled FRP tube by modifying Teng et al.'s original model (Teng et al. 2007b) to account for the biaxial-stress state in FRP tube and Poisson's effect. The axial strain is obtained by:

$$\varepsilon_c = \frac{(\Phi - \Phi K \varepsilon_h) \varepsilon_{co}}{1 - \Phi K k_1 v_l \varepsilon_{co}}$$
(7.19)

$$\Phi = 0.85\{[1 + 0.75(\frac{-\varepsilon_h}{\varepsilon_{co}})]^{0.7} - \exp[-7 \cdot (\frac{-\varepsilon_h}{\varepsilon_{co}})]\}$$
(7.20)

$$K = \frac{8}{f_c} \cdot \frac{E_h t_o}{(D_o / 2) \cdot (v_l v_h - 1)}$$
(7.21)

where Φ and *K* are parameters and k_1 is a converting factor taken as 0.8 for experiments in Chapters 3 and 4. More details of the dilation model of concrete fully filled FRP wrap/tube could be found in Teng et al. (2007b) and Chapter 6.

7.3.2 Dilation model for concrete-filled double-skin FRP tubes

The dilation property of CFDST is probably affected by confinement stiffness ($\rho_{\rm K}$) and void ratio (ϕ) defined later in this section. After collecting the available existing experimental results for doubleskin tubes and corresponding fully filled tubes (FRP wrap/tube as outer tube, carbon steel or stainless steel as inner tube, Wong et al. 2008; Fanggi and Ozbakkaloglu 2013; Chapters 3 and 4), the relationship between the ratio of ultimate axial strain ratio of CFDST to that of CFFT ($\varepsilon_{cu,CFDST}/\varepsilon_{cu,CFFT}$) and confinement stiffness ($\rho_{\rm K}$) is shown in Fig. 7.7, in which the definition of $\rho_{\rm K}$ is (Teng et al. 2009):

$$\rho_{\kappa} = \frac{2E_{h}t_{o}}{f_{c}^{'} / \varepsilon_{co} \cdot D_{o}}$$
(7.22)

In Fig. 7.7, the ultimate axial strain ratio of CFDST-to-CFFT does not change greatly with the confinement stiffness, if the confinement ratio is larger than 0.02. If the concrete is not sufficiently confined (i.e., $\rho_{K} \leq 0.01$ based on Teng et al. 2009), the ratio becomes higher. However, CFDST investigated in the present study are unlikely to be insufficiently confined as the FRP wrap/tube is thick. Therefore, it is concluded that the effect of confinement stiffness on the dilation of CFDST is similar to that of CFFT.



Fig. 7.7. Effects of confinement stiffness on ultimate axial strain

The relationship between $\varepsilon_{cu,CFDST}/\varepsilon_{cu,CFFT}$ and void ratio (φ) is shown in Fig. 7.8. The ratio increases with increase in void ratio, which is in agreement with the existing studies (Wong et al. 2008; Yu et al. 2010). By regression analysis (Fig. 7.8), the relationship between $\varepsilon_{cu,CFDST}/\varepsilon_{cu,CFFT}$ and φ is expressed as:

$$\varepsilon_{cu,CFDST} = \varepsilon_{cu,CFFT} \left(1 - \varphi\right)^{-0.23} \tag{7.23}$$

$$\varphi = \frac{D_i}{D_o - 2t_o} \tag{7.24}$$

where $\varepsilon_{cu,CFFT}$ can be determined by the previously proposed model for concrete fully filled FRP wraps/tubes (Section 7.3.1). It is necessary to mention that a similar form of the term $(1-\varphi)^{-0.23}$ has been adopted by researchers implicitly (Yu et al. 2010) or explicitly (Fanggi and Ozbakkaloglu 2013) to account for the effect of void ratio on the dilation of CFDST.



Fig. 7.8. Effects of void ratio on ultimate axial strain

The dilation curves of CFDST (FRP as outer tube and stainless steel as inner tube) tested in Chapters 3 and 4 are shown in Fig. 7.9, including predictions based on CFFT (denoted as "Prediction (fully filled)") and modified for CFDST (denoted as "Prediction (double-skin)"). The label for the specimen consists of outer tube type ("G" for GFRP, "C" for CFRP and "B" for BFRP) with its diameter, inner tube type with its diameter and concrete indicator (i.e., "C"). It should be mentioned that seawater and sea sand concrete (SWSSC) was used for these specimens. Past studies (Chen et al. 2017a, b; Chapter 2) indicated that the structural behaviour of SWSSC is similar to that of ordinary Portland cement concrete. As shown in Fig. 7.9, after adopting Eq. (7.21), the prediction shows better agreement with experimental results. For specimens C114-S50-C and C165-S101-C, the predicted ultimate axial strains are much lower than experimental values, which is caused by the fact that the rupture strain of FRP tube in CFDST is larger than the rupture strain obtained from split-disk test. Similar underestimation of ultimate axial strain is also found in fully filled tubes (Chapter 6).

Generally, the predicted hoop-axial strain curves are in good agreement with experimental curves, indicating that the dilation of CFDST can be reasonably predicted by the method initially proposed for CFFT with proper modification.



7.4 Behaviour of steel and FRP tubes in CFDST

7.4.1 Carbon steel tube

The filled concrete could prevent or postpone the buckling of inner steel tube. Since obvious yielding plateau exists in carbon steel, it is assumed that the load carried by inner carbon steel tube increases

linearly until the yield capacity. After reaching the yield strain, the load carried by carbon steel tube is assumed to keep constant. This assumption was also widely adopted in the reported studies (Wong et al. 2008; Ozbakkaloglu and Fanggi 2013; Zhang et al. 2017) in deriving the stress in concrete in double-skin tubes.

7.4.2 Stainless steel tube

As compared to carbon steel, common stainless steels (SS) exhibit substantial strain hardening without obvious yielding plateau. It is assumed that the load-strain behaviour of SS tube in a CFDST is same as that of an unfilled circular hollow section (CHS). After reaching the ultimate capacity of CHS, the load carried by SS tube is assumed to keep constant to account for the beneficial effect of in-filled concrete on delaying the tube buckling.

Experimental study (Chapter 5) indicated that the design codes provide underestimation for the ultimate capacity of CHS and the continuous strength method (CSM, Gardner and Theofanous 2008; Abdella 2006) is more suitable for materials exhibiting a high degree of strain hardening, such as stainless steel. CSM was proposed to estimate the plastic local buckling capacity of stainless steel tube with the consideration of strain hardening. The ultimate strain (ε_{LB}) is determined by empirical formula:

$$\frac{\varepsilon_{LB}}{\varepsilon_0} = \frac{0.18}{\lambda_c^{1.24+1.70\lambda_c}}$$
(7.25)

$$\lambda_{c} = \frac{\sqrt{3(1 - v_{s}^{2})}(D_{o} - t_{o})f_{0.2}}{2E_{s}t_{o}}$$
(7.26)

$$\varepsilon_0 = f_{0.2} / E_s \tag{7.27}$$

where λ_c is cross-section slenderness for CHS and v_s is Poisson's ratio of stainless steel. The stressstrain curve up to the ultimate strain could be obtained from the stress-strain model of stainless steel (i.e., Rasmussen's model (2003)).

A comparison between the predicted and experimental curves of CHS tested in Chapters 3 and 4 is shown in Fig. 7.10. The experimental capacity of SS tubes with a diameter of 50 mm and a length of 400 mm is lower than the predication, as shown in Fig. 7.10(a). This is most likely due to the fact that the influence of the overall buckling for this specimen with a length-to-diameter ratio of 8, which prevented the SS tube to reach strain hardening stage. The predicted capacity of SS tubes with a diameter of 50 mm and a length of 150 mm is lower than the experimental one. This may be due to the very small diameter-to-thickness ratio (D/t = 17) which makes the specimen very stocky. Fig.

7.10(b) shows a good agreement with the test results of hollow SS tubes with different diameters indicating the adequacy of CSM in predicting the load-strain curves of SS tubes.

It should be mentioned that the concrete may prevent or delay the occurrence of overall/local buckling of SS tube and this beneficial effect is only considered by assuming that the load carried by SS tube remains constant after reaching the ultimate capacity. Due to the inward expansion of concrete, the SS tube is under biaxial compression state (axial compression and hoop compression) and its axial strength is probably lower than the uniaxial compressive strength. The current method to estimate the load carried by inner steel tube ignores the effect of in-filled concrete and the effect of biaxial stress state in SS tube.



7.4.3 FRP tube

FRP wrap only has fibers in hoop direction and its contribution in axial direction is negligible. On the other hand, FRP tube could provide strength and stiffness in axial direction as well due to the existence of fibers oriented in multiple directions (e.g., fiber layout of [15/45/75] for FRP tubes in Chapter 3 and 4. The behaviour (i.e., buckling and load-carrying capacity) of FRP outer tube has been studied in Chapter 6. Based on the experimental observation, FRP outer tube in a CFDST behaved similar to that in a CFFT. The stresses (i.e., axial stress σ_l and hoop stress σ_h) in the outer FRP tube is calculated by:

$$\begin{cases}
k_{1}\varepsilon_{c} \\
\varepsilon_{h}
\end{cases} = \begin{bmatrix}
\frac{1}{E_{l}} & \frac{\nu_{h}}{E_{h}} \\
\frac{\nu_{l}}{E_{l}} & \frac{1}{E_{h}}
\end{bmatrix} \begin{cases}
\sigma_{l} \\
\sigma_{h}
\end{cases}$$
(7.28)

where ε_c and ε_h are determined by the dilation model in Section 7.3, k_1 is a converting factor to transfer the strain obtained by LVDTs to the strain obtained by strain gauges, which is taken as 0.8 for the specimens in Li et al (2016a, b). The buckling strain and the load shared by FRP outer tube are estimated by the same method proposed in Chapter 6.

Theoretically speaking, FRP outer tube is under a biaxial stress state of axial compression and hoop tension, whereas the inner tube is under axial compression and hoop compression. The in-filled concrete may postpone the occurrence of the FRP tube buckling. Due to the lack of enough experiments, it is still assumed that the behaviour of inner FRP tube is same as that of outer FRP tube. The method proposed to predict the buckling of outer FRP tube (Chapter 6) is adopted for inner FRP tube. Based on the maximum strain criteria, the buckling strain of inner FRP tube is:

$$\varepsilon_{cb} = \frac{k_2}{k_1} \frac{f_{ul}}{E_l} \tag{7.29}$$

$$k_2 = \min\{4.12\rho_K + 1, 2\} \tag{7.30}$$

where k_2 is a factor to consider the beneficial effect of in-filled concrete to postpone the FRP tube buckling. The relationship between k_2 and ρ_K for both outer and inner FRP tubes is shown in Fig. 7.11, in which Eq. (7.30) is presented as dashed lines. Generally, the buckling of inner FRP tube happens slightly earlier than that of the outer FRP tube as the biaxial stress states are different in these tubes. Nevertheless, Eq. (7.30) could still provide a reasonable approximation on the buckling strain of both inner and outer FRP tubes.



Fig. 7.11. Relationship between k_2 and $\rho_{\rm K}$

The calculation of load carried by inner FRP tube is similar to that of outer FRP tube introduced in Chapter 6 except that the Poisson's effect is ignored in calculating the axial stress in FRP tube due to the difficulty in estimating the hoop compressive stress in the inner tube. Since the inner FRP tube only contributes a small part of the total load carried by a specimen and the hoop compressive stress is much lower than the axial stress, this simplification does not significantly affect the prediction accuracy. Before tube buckling, the load shared by inner FRP tube increases linearly with the increase of axial strain until reaching its ultimate capacity (N_{fui}). After tube buckling, the carried load drops instantly and then reduces gradually until reaching zero at the failure of the specimen (i.e., outer FRP tube rupture in hoop direction). Therefore, the load carried by inner FRP tube (N_{fi}) is:

$$N_{fi} = \begin{cases} \min\{A_i E_l k_1 \varepsilon_c, N_{fui}\} & \text{if } \varepsilon_c \le \varepsilon_{cb} \\ (1 - \eta) N_{fui} k_3 & \text{if } \varepsilon_c > \varepsilon_{cb} \end{cases}$$
(7.31)

$$N_{fui} = A_i f_{ul} \tag{7.32}$$

$$k_3 = 1 - \frac{\varepsilon_h - \varepsilon_{hb}}{\varepsilon_{hu} - \varepsilon_{hb}} \tag{7.33}$$

where f_{ul} is compressive strength of FRP tube, η is a load drop factor taken as 0.5 in this study, ε_h is hoop strain, ε_{hb} is hoop strain corresponding to the predicted tube buckling, ε_{hu} is rupture strain of FRP tube.

7.5 Load-axial strain model

7.5.1 Stresses in annular cross-sectional concrete

The annular concrete in a CFDST is non-uniformly confined, which is the key difference to the concrete in a fully filled tube. The circumferential stress (σ_{θ}) and radial stress (σ_{r}) in the concrete are unequal at a given confining stress (f_i). Therefore, the axial stress distribution through the concrete cross-section is non-uniform. A major contribution of the present study is to explicitly consider this non-uniform confinement by the concept of integration.

The existence of inner tube could help to make the stress distribution in concrete more uniform and to avoid the damage of the concrete inner edge (Wong et al. 2008). In order to consider this effect in calculation of the circumferential and radial stresses in concrete, the inner tube is included and it is assumed that Young's modulus of inner tube is same as that of concrete and inner tube is well in contact with concrete. The assumptions are valid during the late stage of loading process because the inner steel tube is in plastic region and its Young's modulus is greatly reduced, FRP tube is partially damaged and its Young's modulus is in the same order as that of concrete, and the concrete inward expansion is restrained by inner tube. It is necessary to mention that the assumptions make it possible to calculate the stresses in annular concrete by simple formulas.

Based on elasticity theory, for a given cross-section dimension, the stresses (σ_{θ} and σ_{r}) are a function of the confining stress (f_{l}) and the location of the free-body. The annular cross-section is divided into many circular layers (e.g., *N* layers) with finite width and it is assumed that the stress state within

each layer is the same (Fig. 7.12a, b). The circumferential and radial stresses (σ_{θ} and σ_{r}) in ith layer (with radius of r_{i}) could be determined by:

$$\sigma_{\theta} = \frac{f_l R_o^2 ((R_i - t_i)^2 + r_i^2)}{r_i^2 (R_o^2 - (R_i - t_i)^2)}$$
(7.34)

$$\sigma_r = \frac{f_l R_o^2 (r_i^2 - (R_i - t_i)^2)}{r_i^2 (R_o^2 - (R_i - t_i)^2)}$$
(7.35)

$$r_i = R_i + \frac{R_o - R_i}{N}(i - 0.5), \qquad i = 1, 2, ..., N$$
 (7.36)

where R_0 is outer radius of annular concrete, R_i is inner radius of annular concrete, t_i is thickness of inner tube, N is number of layers and $N \ge 10$ could meet the requirement for calculation accuracy. The distribution of circumferential and radial stresses (σ_{θ} and σ_{r}) is conceptually illustrated in Fig. 7.12c, in which the stress in inner tube is presented by dashed lines whereas the stress in concrete is shown by solid line. As shown in Fig. 7.12, σ_{θ} decreases and σ_{r} increases from the inner side to the outer side of concrete.



Fig. 7.12. Stresses in annular cross-sectional concrete

After knowing the stress state (i.e., σ_{θ} and σ_{r}) for each concrete layer at a given confining stress (*f_i*), the axial stress in concrete could be determined by the constitutive model developed in Section 7.2. The load carried by the whole concrete section is obtained by summing the load carried by each layer. The equivalent stress in concrete is then calculated by dividing the load by the cross-sectional area of concrete.

7.5.2 Numerical procedures

The generation of the load-strain curves for concrete-filled double-skin FRP tubes/wraps (CFDST) involves the numerical iterative steps. For a CFDST with FRP outer wrap, the steps are: (1) an initial value is set for hoop strain ε_h and it increases by a small increment until reaching the ultimate hoop strain; (2) confining stress f_l and axial strain ε_c is determined by Eqs. (7.16-18, 23); (3) circumferential

stress σ_{θ} and radial stress σ_{r} in annular concrete is determined by Eqs. (7.34-36) and axial stress in concrete σ_{c} is calculated by the constitutive model in Section 7.2; (4) load carried by inner tube is calculated based on Section 7.4; and (5) total load carried by concrete and inner tube is obtained and the load-axial strain curve is then generated. This process is terminated at the rupture of FRP outer wrap.

If FRP tube is taken as the outer tube, the generation of load-axial strain curve is more complicated due to the consideration of Poisson's effect, biaxial stress state and FRP tube buckling. The numerical procedures are divided into two parts: before and after outer FRP tube buckling. If the outer FRP tube has not buckled, the steps are: (1) an initial value is set for hoop strain ε_h and it increases by a small increment until reaching the ultimate hoop strain; (2) axial strain ε_c is determined by Eqs. (7.19-21, 23); (3) axial stress σ_l and hoop stress σ_h in FRP outer tube is determined by Eq. (7.28) and the load carried by outer FRP tube is then calculated; (4) confining stress is equal to the hoop stress in outer FRP tube ($f_l = \sigma_h$) and the stresses (σ_{θ} , σ_r and σ_c) in concrete are determined by Eqs. (7.34-36) and concrete constitutive model (Section 7.2); (5) load carried by inner tube is calculated based on Section 7.4; and (6) total load carried by the specimen is obtained by summing up the loads carried by outer FRP tube, annular concrete and inner tube, and the load-axial strain curve is then generated. After FRP tube buckling, the FRP outer tube is assumed to behave similar to a FRP wrap except its residual load of FRP tubes is considered (Section 7.4.3). It should be mentioned that at the buckling of outer FRP tube, there is jump of the calculated axial strain and a straight line is adopted to connect the stages of before- and post-buckling. More details of generating the load-axial strain curve could be found in Chapter 6, which was initially proposed for concrete fully filled FRP tubes.

As aforementioned, the axial stress in the ith layer of an annular concrete (Fig. 7.12) could be determined by the constitutive model developed in Section 7.2. After obtaining the circumferential and hoop stresses (σ_{θ} and σ_{r}), the non-uniformly confined concrete strength could be determined by Willam-Warnke model. As shown in Section 7.2.2, the parameters of σ_{m} , θ , and ρ are a function of the three principle stresses (i.e., σ_{θ} , σ_{r} and f_{cc} ') but f_{cc} ' is unknown. A "trial-and-error" concept is adopted in the present study to find the value of f_{cc} . An initial value of f_{cc} ' is set as unconfined concrete strength (f_{c} ') and f_{cc} increases by a very small increment. The parameter ρ is calculated by Eqs. (7.2, 4) and Eqs. (7.1, 3, 7-9) separately. If these two values for ρ are close enough (i.e., difference less than 10⁻⁴), the corresponding f_{cc} ' is taken as the confined concrete strength. It should be mentioned that some programming techniques were adopted in writing the MATLAB (2017) script to accelerate the computing speed. After knowing f_{cc} ', the ultimate axial strain (ε_{cc}) could be calculated by Eqs. (7.10, 13) and the axial stress in concrete could be obtained by Eqs. (7.14-15).

7.6 Verification of the proposed model

7.6.1 CFDST with inner carbon steel tube and outer FRP wrap

Several experiments on axially compressed CFDST with inner carbon steel tube and outer FRP wrap have been reported in the literature (Wong et al. 2008, Yu et al. 2012, Xie et al. 2011, Zhang et al. 2017) and they are used to verify the model described above. The specimens with normal concrete strength (i.e., $f_c' \leq 60$ MPa) were selected for verification and the specimen details are summarized in Table 7.2, in which two identical specimens were tested for each case. The fibers are oriented in hoop direction for all specimens except Zhang et al.'s test, in which the fiber direction is $\pm 80^{\circ}$. Since the axial-to-hoop strength ratio of FRP tube in Zhang et al. (2017) is very low (i.e., 0.14) and the stress-strain curves of concrete have been presented in their paper, this experiment is still categorized as concrete-filled FRP wrap for an easy comparison. As shown in Table 7.2, the void ratio (φ) of the specimens ranges from 0.28 to 0.81 and the strength enhancement ratio (f_{cc}'/f_c') varies from 1.01 to 3.02. The experimental stress-axial strain curves and hoop-axial strain curves presented in Fig. 7.13 were replotted from the curves published in their papers, in which the axial load carried by the inner carbon steel tube has been subtracted. It is necessary to mention that some curves grouped under data source of Wong et al. (2008) were replotted from the same authors' companion paper (Yu et al. 2010) which cited the experiments in Wong et al. (2008).





Fig. 7.13. Comparison of predicted stress-strain curves and hoop-axial strain curves to those of experiments



Data source	Specimen	$D_{\rm o}{}^{\rm a}$	D_{i}	to	ti	$f_{\rm uh}$	Eh	Ø	fc'	$f_{\rm cc}$	Ecu
Wang at al		(mm)	(mm)	(mm)	(mm)	(MPa)	(GPa)	, , , , , , , , , , , , , , , , , , , ,	(MPa)	(MPa)	0.010
(2008)	D37-A2-I	153.2	42.0	0.34	2.30	1826	80.1	0.28	36.7	54.6	0.019
(2008)	D37-A2-II	153.2	42.0	0.34	2.30	1826	80.1	0.28	36.7	49.6	0.016
	D40-B1-I	152.8	76.0	0.17	3.30	1826	80.1	0.50	39.6	41.5	0.015
	D40-B1-II	152.8	76.0	0.17	3.30	1826	80.1	0.50	39.6	40.1	0.014
	D40-B2-I	153.2	76.0	0.34	3.30	1826	80.1	0.50	39.6	56.3	0.022
	D40-B2-II	153.2	76.0	0.34	3.30	1826	80.1	0.50	39.6	55.0	0.018
	D47-B2-I	153.2	76.0	0.34	3.50	1826	80.1	0.50	46.7	60.0	0.022
	D47-B2-II	153.2	76.0	0.34	3.50	1826	80.1	0.50	46.7	55.7	0.015
	D40-B3-I	153.5	76.0	0.51	3.30	1826	80.1	0.50	39.6	68.7	0.023
	D40-B3-II	153.5	76.0	0.51	3.30	1826	80.1	0.50	39.6	67.8	0.024
	D37-C1-I	152.8	88.0	0.17	2.10	1826	80.1	0.58	36.7	42.9	0.017
	D37-C1-II	152.8	88.0	0.17	2.10	1826	80.1	0.58	36.7	41.4	0.013
	D37-C2-I	153.2	88.0	0.34	2.10	1826	80.1	0.58	36.7	55.9	0.024
	D37-C2-II	153.2	88.0	0.34	2.10	1826	80.1	0.58	36.7	52.9	0.019
	D37-C3-I	153.5	88.0	0.51	2.10	1826	80.1	0.58	36.7	69.4	0.024
	D37-C3-II	153.5	88.0	0.51	2.10	1826	80.1	0.58	36.7	69.2	0.028
	D40-D2-I	153.2	115.0	0.34	5.20	1826	80.1	0.75	40.1	N/A ^b	0.030
	D40-D2-II	153.2	115.0	0.34	5.20	1826	80.1	0.75	40.1	N/A^b	0.025
Yu et al.	M1	206.0	140.3	0.34	5.30	1752	89.4	0.68	43.9	57.4	0.018
(2012)	M2	206.0	140.3	0.34	5.30	1752	89.4	0.68	43.9	56.3	0.017
Xie et al.	CC245-W6-I	402.0	245.8	1.02	8.00	1826	80.1	0.61	29.3	48.0	0.029
(2011)	CC245-W6-II	402.0	245.8	1.02	8.00	1826	80.1	0.61	29.3	52.1	0.029
	CC325-W6-I	402.0	323.6	1.02	9.30	1826	80.1	0.81	37.3	76.2	0.026
	CC325-W6-II	402.0	323.6	1.02	9.30	1826	80.1	0.81	37.3	65.4	0.026
Zhang et al.	D54-2FW-M	204.4	159.0	2.20	5.00	684	45.9	0.80	54.1	90.7	0.019
(2017)	D54-4FW-M	209.4	159.0	4.70	5.00	684	45.9	0.80	54.1	121.1	0.028
	D40-6FW-M	312.0	219.0	6.00	6.00	676	43.6	0.73	40.9	123.4	0.037

Table 7.2. Details of CFDST (inner carbon steel tube and outer FRP wrap) for model verification

Past research (Lam and Teng 2004) has proved that the rupture strain of FRP wrap (ε_{hu}) in a column is much smaller than the ultimate strain obtained by flat coupon test (ε_f). A relationship between ε_{hu} and ε_f was recently proposed by Lim and Ozbakkaloglu (2014b), which is expressed as:

$$\varepsilon_{hu} = k_{\varepsilon f} \varepsilon_f \tag{7.37}$$

$$k_{ef} = 0.9 - 2.3f'_{e} \times 10^{-3} - 0.75E_{h} \times 10^{-6}$$
(7.38)

where f_c ' and E_h are in MPa, and 100,000 MPa $\leq E_h \leq$ 640,000 MPa. Eqs. (7.37-38) were adopted in the present study to estimate the rupture strain of FRP wrap. A comparison between the predicted and experimental stress-axial strain curves and hoop-axial strain curves (if applicable) is shown in Fig. 7.13. The predicted stress is an equivalent stress in concrete, which is the load carried by concrete divided by its cross-sectional area. As indicated by Fig. 7.13, very good accuracy is achieved by the proposed analysis-oriented model. Fig. 7.14 presents comparisons of the predicted and experimental ultimate capacities and ultimate axial strain. The average and coefficient of variation (COV) of the ratios of predicted-to-experimental capacity are 1.02 and 0.08 respectively and those for ultimate axial strain are 1.02 and 0.14 respectively. Generally speaking, the proposed model could accurately predict the structural behaviour (i.e., dilation and stress-strain behaviour) of CFDST with carbon steel as inner tube and FRP wrap as outer tube.

7.6.2 CFDST with inner SS tube and outer FRP tube

Only six specimens were reported by the author (Chapters 3 and 4) on CFDST with stainless steel as inner tube and FRP as outer tube. Specimen details of CFDST are listed in Table 7.3, in which f_{ul} and E_l are longitudinal compressive strength and Young's modulus of FRP, v_l and v_h are Poisson's ratio in longitudinal and hoop directions, $f_{0.2}$ is 0.2% proof stress of stainless steel. Seawater and sea sand concrete (SWSSC) was used for the specimens and past studies (Chen et al. 2007) indicated that the material properties of SWSSC are similar to those of ordinary Portland cement concrete. Comparisons between the predicted and experimental load-axial strain curves and hoop-axial strain curves are shown in Fig. 7.15. Table 7.4 summarizes the predicted ultimate capacity, ultimate axial strain and hoop rupture strain and they were compared to the experimental values. For CFDST with GFRP outer tube, the predicted load-axial strain curves are slightly higher than the experimental curves prior to FRP tube buckling, whilst the prediction is conservative after tube buckling. The predicted ultimate axial strain and ultimate capacity are much lower than experimental values for specimen C165-S101-C. Similar observation is also found for concrete fully-filled CFRP tube (C165-C) in Chapter 6. The reason is likely to be that the hoop strength of CFRP outer tube in the column is much higher than that obtained by "split-disk test", which could be validated by the fact that the predicted rupture strain of C165-S101-C is only 2/3 of the measured rupture strain. Nevertheless, the trend of the predicted curves is in good agreement with the experimental data. Generally speaking, the prediction agrees well with the experimental results indicating the reasonability of the proposed model.

Chapter 7 Analysis-oriented load-strain model for concrete-filled double-skin circular FRP ...

Specimen	Tube	Lube type Dimensions			FRP						SS		SWSSC			
	Outer	Inner	D _o (mm)	t _o (mm)	D _i (mm)	<i>t</i> _i (mm)	φ	ful (MPa)	f _{uh} (MPa)	E_l (GPa)	E _h (GPa)	Vl	\mathcal{V}_{h}	Es (GPa)	f _{0.2} (MPa)	fc' (MPa)
G114-S50-C	GFRP	SS	114.8	2.91	47.9	2.82	0.44	195	309	21.5	25.2	0.27	0.44	195	306.8	31.4
G165-S101-C	GFRP	SS	158.0	2.92	101.8	2.91	0.67	195	309	21.5	25.2	0.27	0.44	195	324.4	31.4
C114-S50-C	CFRP	SS	114.6	2.75	50.9	3.07	0.47	163	593	40.0	66.7	0.26	0.52	195	228.2	39.4
C165-S101-C	CFRP	SS	158.1	2.79	101.9	2.79	0.67	163	593	40.0	66.7	0.26	0.52	195	225.7	39.4
В114-S50-С	BFRP	SS	114.5	2.78	50.9	3.07	0.47	114	331	23.9	24.3	0.29	0.30	195	228.2	32.8
B165-S101-C	BFRP	SS	157.7	2.71	101.9	2.79	0.67	114	331	23.9	24.3	0.29	0.30	195	225.7	32.8
G114-G50-C	GFRP	GFRP	114.8	2.91	51.1	3.07	0.47	195	309	21.5	25.2	0.27	0.44			31.4
G165-G101-C	GFRP	GFRP	158.0	2.92	100.1	3.13	0.66	195	309	21.5	25.2	0.27	0.44			31.4
C114-C50-C	CFRP	CFRP	114.6	2.75	50.5	2.81	0.46	163	593	40.0	66.7	0.26	0.52			39.4
C165-C101-C	CFRP	CFRP	158.1	2.79	99.9	2.81	0.65	163	593	40.0	66.7	0.26	0.52	IN	/A	39.4
B114-B50-C	BFRP	BFRP	114.5	2.78	50.0	2.71	0.46	114	331	23.9	24.3	0.29	0.30			32.8
B165-B101-C	BFRP	BFRP	157.7	2.71	100.0	2.92	0.66	114	331	23.9	24.3	0.29	0.30			32.8

Table 7.3. Details of seawater and sea sand concrete-filled double-skin FRP tubes in Chapters 3 and 4



Fig. 7.15. Comparison of predicted load-strain curves and hoop-axial strain curves to those of experiments (DSTCs with inner SS tube and outer FRP tube)



Fig. 7.16. Comparison of predicted load-strain curves and hoop-axial strain curves to those of experiments (DSTCs with inner and outer FRP tubes)

Specimen	Ult	imate capac	ity	Ulti	mate axi	al strain	Hoop rupture strain			
	Exp. (kN)	Pre. (kN)	Pre./Exp.	Exp.	Pre.	Pre./Exp.	Exp.	Pre.	Pre./Exp.	
G114-S50-C	872	865	0.99	0.033	0.039	1.18	0.011	0.015	1.33	
G165-S101-C	1301	1157	0.89	0.036	0.034	0.95	0.014	0.015	1.04	
C114-S50-C	1375	1252	0.91	0.052	0.044	0.85	0.014	0.010	0.71	
C165-S101-C	1698	1478	0.87	0.058	0.038	0.65	0.015	0.010	0.66	
В114-S50-С	884	851	0.96	0.040	0.039	0.98	0.016	0.015	0.92	
B165-S101-C	1053	1027	0.98	0.035	0.033	0.94	0.019	0.015	0.77	
Mean			0.93			0.93			0.91	
COV			0.05			0.17			0.25	

Table 7.4. Prediction of ultimate capacity, ultimate axial strain and hoop rupture strain (CFDST with inner SS tube and outer FRP tube)

Table 7.5. Prediction of ultimate capacity, ultimate axial strain and hoop rupture strain (CFDST with inner and outer FRP tubes)

Spacimon	l	Iltimate load	1	Ulti	mate axi	al strain	Hoop rupture strain			
specifien	Exp. (kN)	Pre. (kN)	Pre./Exp.	Exp.	Pre.	Pre./Exp.	Exp.	Pre.	Pre./Exp.	
G114-S50-C	797	742	0.93	0.033	0.027	0.81	0.012	0.015	1.19	
G165-S101-C	882	1021	1.16	0.032	0.024	0.74	0.008	0.015	1.73	
C114-S50-C	1175	1107	0.94	0.060	0.044	0.74	0.014	0.010	0.69	
C165-S101-C	1219	1276	1.05	N/A	0.037	N/A	N/A	0.010	N/A	
B114-S50-C	651	658	1.01	N/A	0.027	N/A	N/A	0.015	N/A	
B165-S101-C	703	777	1.10	N/A	0.023	N/A	N/A	0.015	N/A	
Mean			1.03			0.76			1.20	
COV			0.08			0.04			0.35	

7.6.3 CFDST with FRP as both inner and outer tubes

CFDST with FRP as both inner and outer tubes were tested in Chapters 3 and 4 and the specimen details are shown in Table 7.3, in which the properties of inner and outer FRP tubes are the same. Comparisons of the predicted and experimental load-axial strain curves, hoop-axial strain curves, ultimate capacity, ultimate axial strain and hoop rupture strain are shown in Fig. 7.16 and Table 7.5. As shown in Fig. 7.16, the hoop strains of some specimens (e.g., C165-C101-C, B114-B50-C and B165-B101-C) do not increase during the late stage of loading process and a smooth load decrease is observed for these specimens. The reason is that the inner FRP tube was seriously damaged and the inward expansion of concrete was not effectively restrained. These specimens were not failed by outer FRP tube rupture in hoop direction. However, the proposed model assumes that the outer FRP tube could reach its rupture strength. Therefore, the feature of ineffective restraint on sandwich concrete in FRP-concrete-FRP specimens could not be captured by the proposed model and the predicted capacity is higher. Nevertheless, the overestimation for ultimate capacity is less than 20% and the predicted curves still match well with the experimental curves except in the post-peak region.

7.7 Conclusions

This chapter proposed an analysis-oriented load-axial strain model for concrete-filled double-skin FRP circular tubes under axial compression, in which the non-uniform confinement in annular concrete is properly considered. The following conclusions are made.

(1) A constitutive model is developed for non-uniformly FRP-confined concrete.

(2) The dilation model for concrete fully filled tubes is properly modified to predict the hoop-axial strain behaviour of concrete-filled double-skin tubes.

(3) Continuous strength method (CSM, Gardner and Theofanous 2008) is capable of estimating the stress-strain behaviour of stainless steel inner tube. The buckling of inner FRP tube is predicted by maximum strain criteria with proper consideration of the beneficial effect from in-filled concrete. A simplified method is proposed to estimate the load carried by inner FRP tube by ignoring the insignificant Poisson's effect.

(4) Non-uniform stress distribution in concrete is explicitly considered in developing the load-axial strain model by dividing the annular cross-section into multiple circular layers. The proposed analysis-oriented model for concrete-filled double-skin FRP tubes reached reasonable prediction of experimental load-axial strain curves from a wide range of literatures.

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References

Abdella, K. (2006), Inversion of a full-range stress-strain relation for stainless steel alloys, International Journal of Non-Linear Mechanics, 41(3), 456-463.

ACI 318-11. (2011), Building code requirements for structural concrete and commentary, American Concrete Institute, Farmington Hills, MI.

Bank, L.C. (2006), Composites for construction: structural design with FRP materials, John Wiley and Sons, Hoboken, New Jersey.

Becque, J., Patnaik, A.K., and Rizkalla, S.H. (2003), Analytical models for concrete confined with FRP tubes, Journal of Composites for Construction, 7(1), 31-38.

Carreira, D.J., and Chu, K.-H. (1985), Stress-strain relationship for plain concrete in compression, ACI Journal, 82(6), 797-804.

Chen, G.M., He, Z.B., Jiang, T., Chen, J.F., and Teng, J.G. (2017), Axial compression tests on FRPconfined seawater/sea-sand concrete, 6th Asia-Pacific Conference on FRP in Structures.

Chen, W.-F., and Saleeb, A.F. (1994), Constitutive equations for engineering materials: elasticity and modelling, Elsevier.

Cordon, W.A., and Gillespie, H.A. (1963), Variables in concrete aggregates and Portland cement paste which influence the strength of concrete, Journal Proceedings, American Concrete Institute, 60(8), 1029-1052.

Fam, A.Z., and Rizkalla, S.H. (2001a), Behavior of axially loaded concrete-filled circular FRP tubes, ACI Structural Journal, 98(3), 280-289.

Fam, A.Z., and Rizkalla, S.H. (2001b), Confinement model for axially loaded concrete confined by circular fiber-reinforced polymer tubes, ACI Structural Journal, 98(4), 451-461.

Fanggi, B.A.L., and Ozbakkaloglu, T. (2013), Compressive behavior of aramid FRP–HSC–steel double-skin tubular columns, Construction and Building Materials, 48, 554-565.

Fanggi, B.L., and Ozbakkaloglu, T. (2015), Behavior of hollow and concrete-filled FRP-HSC and FRP-HSC-steel composite columns subjected to concentric compression, Advances in Structural Engineering, 18(5), 715-738.

Gardner, L., and Theofanous, M. (2008), Discrete and continuous treatment of local buckling in stainless steel elements, Journal of Constructional Steel Research, 64(11), 1207-1216.

Gerstle, K.H. (1981a), Simple formulation of biaxial concrete behavior, Journal Proceedings, American Concrete Institute, 78(1), 62-68.

Gerstle, K.H. (1981b), Simple formulation of triaxial concrete behavior, Journal Proceedings, American Concrete Institute, 78(5), 382-387.

Han L.H., Huang H. and Zhao X.L. (2009), Analytical behaviour of concrete-filled double skin steel tubular (CFDST) beam-columns under cyclic loading, Thin-Walled Structures, 47(6-7), 668-680.

Hollaway, L.C., and Teng, J.-G. (2008), Strengthening and rehabilitation of civil infrastructures using fibre-reinforced polymer (FRP) composites, Elsevier.

Imran, I., and Pantazopoulou, S. (1996), Experimental study of plain concrete under triaxial stress, ACI Materials Journal, 93(6), 589-601.

Jiang, T., and Teng, J.G. (2007), Analysis-oriented stress–strain models for FRP–confined concrete, Engineering Structures, 29(11), 2968-2986.

Kupfer, H., Hilsdorf, H.K., and Rusch, H. (1969), Behavior of concrete under biaxial stresses, Journal Proceedings, American Concrete Institute, 66(8), 656-666.

Lam, L., and Teng, J.G. (2004), Ultimate condition of fiber reinforced polymer-confined concrete, Journal of Composites for Construction, 8(6), 539-548.

Lan, S., and Guo, Z. (1997), Experimental investigation of multiaxial compressive strength of concrete under different stress paths, ACI Materials Journal, 94(5), 427-434.

Lee, S.-K., Song, Y.-C., and Han, S.-H. (2004), Biaxial behavior of plain concrete of nuclear containment building, Nuclear Engineering and Design, 227(2), 143-153.

Li, Q., and Ansari, F. (1999), Mechanics of damage and constitutive relationships for high-strength concrete in triaxial compression, Journal of Engineering Mechanics, 125(1), 1-10.

Li, Q., and Ansari, F. (2000), High-strength concrete in triaxial compression by different sizes of specimens, ACI Materials Journal, 97(6), 684-689.

Li, Y.L., Zhao, X.L., Raman Singh, R. K., and Al-Saadi, S. (2016a), Experimental study on seawater and sea sand concrete filled GFRP and stainless steel tubular stub columns, Thin-Walled Structures, 106, 390-406.

Li, Y.L., Zhao, X.L., Raman Singh, R. K., and Al-Saadi, S. (2016b), Tests on seawater and sea sand concrete-filled CFRP, BFRP and stainless steel tubular stub columns, Thin-Walled Structures, 108, 163-184.

Li, Y.L., Zhao, X.L., Singh Raman, R. K., and Al-Saadi, S. (2018a), Thermal and mechanical properties of alkali-activated slag paste, mortar and concrete utilising seawater and sea sand, Construction and Building Materials, 159, 704-724.

Li, Y.L., Teng, J.G., Zhao, X.L., and Singh Raman, R. K. (2018b), Theoretical model for seawater and sea sand concrete-filled circular FRP tubular stub columns under axial compression, Engineering Structures, 160, 71-84.

Li, Y.L., Zhao, X.L., Singh Raman, R.K., and Yu, X. (2018c), Axial compression tests on seawater and sea sand concrete-filled double-skin stainless steel tubes, Engineering Structures. (under review)

Li, Y.L., Zhao, X.L., and Singh Raman, R.K. (2018d), Durability of seawater and sea sand concrete-filled FRP tubes in artificial seawater, Construction and Building Materials. (under review)

Lim, J.C., and Ozbakkaloglu, T. (2014a), Lateral strain-to-axial strain relationship of confined concrete, Journal of Structural Engineering, 141(5), 04014141.

Lim, J.C., and Ozbakkaloglu, T. (2014b), Confinement model for FRP-confined high-strength concrete, Journal of Composites for Construction, 18(4), 04013058.

Lim, J.C., and Ozbakkaloglu, T. (2015), Investigation of the influence of the application path of confining pressure: Tests on actively confined and FRP-confined concretes, Journal of Structural Engineering, 141(8), 04014203.

Lu, X., and Hsu, C.-T.T. (2007), Stress-strain relations of high-strength concrete under triaxial compression, Journal of Materials in Civil Engineering, 19(3), 261-268.

Mander, J.B., Priestley, M.J., and Park, R. (1988), Theoretical stress-strain model for confined concrete, Journal of Structural Engineering, 114(8), 1804-1826.

MATLAB2017b. (2017), MathWorks.

Mills, L.L., and Zimmerman, R.M. (1970), Compressive strength of plain concrete under multiaxial loading conditions, Journal Proceedings, American Concrete Institute, 67(10), 802-807.

Mirmiran, A., and Shahawy, M. (1997), Dilation characteristics of confined concrete, Mechanics of Cohesive - Frictional Materials, 2(3), 237-249.

Nelissen, L. (1972), Biaxial testing of normal concrete: Stevin-Laboratory of the Department of Civil Engineering, Delft University of Technology; Reijswik (ZH): Institute TNO for Building Materials and Building Structures.

Ozbakkaloglu, T., Fanggi, B.A.L., and Zheng, J. (2016), Confinement model for concrete in circular and square FRP–concrete–steel double-skin composite columns, Materials and Design, 96, 458-469.

Ozbakkaloglu, T., and Fanggi, B.L. (2013), Axial compressive behavior of FRP-concrete-steel double-skin tubular columns made of normal-and high-strength concrete, Journal of Composites for Construction, 18(1), 04013027.

Ozbakkaloglu, T., Lim, J.C., and Vincent, T. (2013), FRP-confined concrete in circular sections: Review and assessment of stress–strain models, Engineering Structures, 49, 1068-1088.

Park, J.W. and Choi, S.M. (2013), Structural behavior of CFRP strengthened concrete-filled steel tubes columns under axial compression loads, Steel and Composite Structures, 14(5), 453-472.

Popovics, S. (1973), A numerical approach to the complete stress-strain curve of concrete, Cement and Concrete Research, 3(5), 583-599.

Rasmussen, K.J. (2003), Full-range stress-strain curves for stainless steel alloys, Journal of Constructional Steel Research, 59(1), 47-61.

Ren, X., Yang, W., Zhou, Y., and Li, J. (2008), Behavior of high-performance concrete under uniaxial and biaxial loading, ACI Materials Journal, 105(6), 548-558.

Richart, F.E., Brandtzaeg, A., and Brown, R.L. (1928), A study of the failure of concrete under combined compressive stresses, University of Illinois at Urbana Champaign, College of Engineering, Engineering Experiment Station.

Sfer, D., Carol, I., Gettu, R., and Etse, G. (2002), Study of the behavior of concrete under triaxial compression, Journal of Engineering Mechanics, 128(2), 156-163.

Spoelstra, M.R., and Monti, G. (1999), FRP-confined concrete model, Journal of Composites for Construction, 3(3), 143-150.

Tasuji, M.E., and Nilson, A.H. (1978), Stress-strain response and fracture of concrete in biaxial loading, Journal Proceedings, American Concrete Institute, 75(7), 306-312.

Teng, J., Yu, T., Wong, Y., and Dong, S. (2007a), Hybrid FRP–concrete–steel tubular columns: concept and behavior, Construction and Building Materials, 21(4), 846-854.

Teng, J.G., Huang, Y. L., Lam, L., and Ye, L.P. (2007b), Theoretical model for fiber-reinforced polymer-confined concrete, Journal of Composites for Construction, 11(2), 201-210.

Teng, J.G., Jiang, T., Lam, L., and Luo, Y.Z. (2009), Refinement of a design-oriented stress–strain model for FRP-confined concrete, Journal of Composites for Construction, 13(4), 269-278.

Teng, J.G., and Lam, L. (2004), Behavior and modeling of fiber reinforced polymer-confined concrete, Journal of Structural Engineering, 130(11), 1713-1723.

Wang, C.-Z., Guo, Z.-H., and Zhang, X.-Q. (1987), Experimental investigation of biaxial and triaxial compressive concrete strength, ACI Materials Journal, 84(2), 92-100.

Wang, J., Liu, W., Zhou, D., Zhu, L. and Fang, H. (2014), Mechanical behaviour of concrete filled double skin steel tubular stub columns confined by FRP under axial compression, Steel and Composite Structures, 17(4), 431-452.

Wang, Z., Zhao, X.-L., Xian, G., Wu, G., Singh Raman, R.S., and Al-Saadi, S. (2018), Effect of sustained load and seawater and sea sand concrete environment on durability of basalt-and glass-fibre reinforced polymer (B/GFRP) bars, Corrosion Science, 138, 200-218.

Wang, Z., Zhao, X.-L., Xian, G., Wu, G., Singh Raman, R.K., Al-Saadi, S., and Haque, A. (2017), Long-term durability of basalt-and glass-fibre reinforced polymer (BFRP/GFRP) bars in seawater and sea sand concrete environment, Construction and Building Materials, 139, 467-489.

Willam, K., and Warnke, E. (1975), Constitutive Model for the Triaxial Behavior of Concrete, Proceedings of International Association for Bridge and Structural Engineering, Vol. 19, ISMES, Bergamo, Italy.

Wong, Y., Yu, T., Teng, J., and Dong, S. (2008), Behavior of FRP-confined concrete in annular section columns, Composites Part B: Engineering, 39(3), 451-466.

Xie, J., Elwi, A., and MacGregor, J. (1995), Mechanical properties of three high-strength concretes containing silica fume, ACI Materials Journal, 92(2), 135-145.

Xie, P., Yu, T., Wong, Y., and Teng, J. (2011), Compressive behavior of large-scale hybrid FRPconcrete-steel double-skin tubular columns, Advanced Materials Research, Trans Tech Publications, 243, 1138-1144.

Yu, T., Teng, J., and Wong, Y. (2010), Stress-strain behavior of concrete in hybrid FRP-concretesteel double-skin tubular columns, Journal of Structural Engineering, 136(4), 379-389.

Yu, T., Zhang, B., Cao, Y., and Teng, J. (2012), Behavior of hybrid FRP-concrete-steel double-skin tubular columns subjected to cyclic axial compression, Thin-Walled Structures, 61, 196-203.

Zhang, B., Teng, J.G., and Yu, T. (2017), Compressive behavior of double-skin tubular columns with high-strength concrete and a filament-wound FRP tube, Journal of Composites for Construction, 21(5), 04017029.

Zhao, X.-L. (2013), FRP-strengthened metallic structures, CRC Press, London.

Zhao, X.L., Grzebieta, R.H. and Elchalakani, M. (2002), Tests of concrete-filled double skin (CHS Outer and CHS Inner) composite stub columns, Steel and Composite Structures, 2(2), 129-146.

Zhao, X.L. and Han, L.H. (2006), Double skin composite construction, Progress in Structural Engineering and Materials, 8(3), 93-102.

Han L.H, Huang H. and Zhao X.L. (2009), Analytical behaviour of concrete-filled double skin steel tubular (CFDST) beam-columns under cyclic loading, Thin-Walled Structures, 47(6-7), 668-680.

Zhao, X.L., Tong, L.W. and Yang, X.Y. (2010), CFDST stub columns subjected to large deformation axial loading, Engineering Structures, 32(3), 692-703.

Chapter 8

Mechanical properties of seawater and sea sand concretefilled FRP tubes in artificial seawater

Abstract

Application of the hybrid structure system utilizing seawater and sea sand concrete (SWSSC) and fiber reinforced polymer (FRP) is promising as it is resource beneficial as well as FRPs are corrosion resistant. This chapter presents an experimental study on the durability of SWSSC-filled glass/carbon/basalt-FRP tubes exposed artificial seawater (3.5% salt solution) at 40 °C for up to 6 months. The mechanical properties of conditioned and unconditioned plain concrete, FRPs, and concrete-filled FRP tubes (fully filled and double-skin tubes) were measured by the means of compressive test and split-disk test. Compressive strength of SWSSC did not degrade after exposure and obvious hoop strength degradation was observed for GFRP and BFRP. Mechanical properties, such as strength and ultimate axial strain of SWSSC-filled FRP tubes were reduced by environmental aging and the major mechanism is the hoop strength deterioration of FRP. Load-axial strain curves and ultimate capacity of SWSSC-filled FRP tubes were predicted by using an existing method with proper modification to account for the environmental effects. The predictions were in good agreement with experimental results. Finally, the environmental factor specified in current guidelines for FRP degradation was assessed using the data obtained in this study.

Keywords

Durability; Seawater and sea sand concrete (SWSSC); Fiber reinforced polymer (FRP); Concrete-filled FRP tube; Compressive test; Confinement

8.1 Introduction

As an advancing material, fiber reinforced polymer (FRP) composites have been increasingly applied in civil engineering, such as retrofitting and new constructions (Bank 2006). Concrete-filled FRP tubes, i.e., fully filled and double-skin tubes, offer the advantages of high strength, large capacity and desirable corrosion resistance. Traditional Portland cement concrete industry consumes larger quantity of natural resources (i.e., fresh water and river sand) and exacerbates environmental issues (e.g., CO₂ emission from cement production) (Provis and Bernal 2014). It is a promising structural form to utilize seawater and sea sand concrete (SWSSC) with the reinforcement of FRP composite (Teng et al. 2011) due to the superior corrosion resistance of FRP than carbon steel when subjecting to chloride solutions. Several studies (Li et al. 2016a, b; Chen et al. 2017a, b) have been conducted on the short-term structural behaviours of SWSSC-filled FRP tubes, which indicate that the concrete type does not significantly affect the short-term structural behaviour of concrete-filled FRP tubes. However, the long-term (20-50 years) performance of SWSSC-filled FRP tube may be a major concern. The environmental factors that could deteriorate the mechanical properties of concrete-filled FRP tubes/wraps include moisture, alkalinity, extreme temperature, thermal cycling (e.g., heating and cooling), freezing and thawing, creep, fatigue and exposure to ultraviolet radiation (Karbhari 2007). In order to reasonably assess the durability performance of SWSSC-filled FRP tube, the effect of environmental factors on individual components (i.e., SWSSC and FRP) and on the interaction between them (e.g., confinement effect) should be well understood.

The usage of seawater and sea sand in concrete construction was recently reviewed by Xiao et al. (2017). The studies on the durability of seawater and sea sand concrete (e.g., Huiguang et al. 2011, Nishida et al. 2015, Li et al. 2018a) mainly focused on chloride diffusion and penetration, carbonisation, freeze-thaw resistance, thermal behaviour, alkali-aggregate reaction and long-term performance in marine environment. Specifically, Mohammed et al. (2004) found that the use of seawater as mixing water does not cause the deterioration of concrete strength after 15 years of exposure in a tidal pool.

The durability of FRP composites subjected to various environmental conditions has been extensively investigated in recent decades by accelerated degradation test (GangaRao et al. 2006; Chen et al. 2007). The degradation of FRP could result from the deterioration of matrix, fiber and the interphase between them (Nkurunziza et al. 2005). The strength degradation of GFRP and BFRP bars exposed to simulated seawater and sea sand concrete (SWSSC) solution and preloads was studied by Wang et al. (2017; 2018). After 63-day aging in SWSSC at 40 °C, the respective strength losses of BFRP and GFRP bars were 9.8% and 5.9% for unstressed bars, and 18.3% and 25.9% for preloaded bars. Robert and Fam (2012) investigated the long-term performance of GFRP tubes filled with concrete and subjected to salt solution. The hoop strength reduction of GFRP tube after conditioning at 50 °C for 12 months was about 20%. In order to correlate the accelerated degradation test data to FRP structure's real performance in natural environments, many prediction models (e.g., Litherland (1981), Katsuki and Uomoto (1995)) have been proposed but no consensus has been reached.

As compared to FRP composites, the studies on the durability of concrete-filled FRP wraps/tubes are rather limited (Micelli et al. 2014). The effects of ultraviolet radiation, freezing-thawing, heating-cooling and wet-dry cycles on the long term performance of concrete-filled FRP wraps were researched by Silva (2007), Touganji and Balaguru (1998), Toutanji (1999), Toutanji and Saafi (2001), Boumarafi et al. (2015), Erdil et al. (2013) and El-Hacha et al. (2010). The durability of FRP-confined concrete subjected to various solution types, such as acid, alkaline or saline solutions, was studied by Micelli and Myers (2008), Kshirsagar et al. (2000), Liu et al. (2002), Eldridge and Fam (2014), and Walker and Karbhari (2007). Besides the accelerated degradation test, the durability of concrete-filled FRP wrap exposed to natural environments (e.g., indoor and outdoor environments)
was also studied by Saenz and Pantelides (2006), Erdil et al. (2013) and Xie et al. (2018). The aging durations in these studies were up to 30 months (Xie et al. 2018). Fig 8.1 summarizes the normalized strength (f_{cc} '/ f_c ') degradation data of concrete-filled FRP wraps subjected to alkaline solution, neutral solution, salt solution, and outdoor environment. Obvious strength deterioration is observed for concrete-filled FRP wraps after the exposures. Kshirsagar et al.'s study (2000) indicated that temperature greatly affects durability performance, whereas the effect of temperature was found to be insignificant in Eldridge and Fam's study (2014). The influence of the thickness of FRP wrap is not obvious in Xie et al.'s experiment (2018).



Fig. 8.1. Summary of some existing aging data for concrete-filled FRP wraps

The literature review suggests that the durability studies mainly focused on concrete-filled FRP wraps, which are commonly adopted for strengthening. However, no study has been conducted on the long-term behaviour of concrete-filled FRP tubes, which offer strength in longitudinal direction. Furthermore, the effects of seawater and sea sand concrete (SWSSC) and seawater on the durability of SWSSC-filled tubes should be well understood before applying them into a marine environment. This chapter presents an experimental study on the durability performance of SWSSC-filled FRP tubes subjected to artificial sea water (3.5% salt solution). Axial compressive tests were conducted on SWSSC-filled FRP tubular stub columns with or without aging. Mechanical properties of plain concrete, FRP alone and SWSSC-filled FRP tubes were investigated and the load-axial strain models for short-term behaviour of SWSSC-filled FRP tube were properly modified to account for the environmental effects.

8.2 Experimental program

8.2.1 Experiment plan and specimens

Accelerated degradation test was conducted to study the durability performance of SWSSC-filled FRP tubes in an artificial seawater environment. Three types of FRP (i.e., glass-FRP, carbon-FRP and basalt-FRP) were investigated in the present study. After curing in air at room temperature for 28

days, the specimens, including plain SWSSC cylinders, FRP rings with SWSSC and SWSSC-filled FRP tubular stub columns, were immersed in artificial seawater (i.e., 3.5% salt solution, ASTM D1141-98) at 40 °C for different durations (1, 3 and 6 months). After each duration, mechanical tests were conducted on these specimens. The test results of unconditioned stub columns, which have been reported in authors' previous publications (Chapters 3 and 4, Li et al. 2016a, b), were also included in this study as the base-line data.

In order to investigate the degradation of FRP alone, short FRP tubes with SWSSC were prepared and immersed in salt solution at 40 °C. Only one temperature (40 °C) was chosen to minimise the number of specimens in the testing program. The reason for choosing 40 °C is because of the temperature in marine environment is most likely be below 40 °C. Furthermore, previous studies by the authors on durability of FRP within seawater sea sand environment (Guo et al. 2018) revealed that significant degradation of FRP occurs at 40 °C, i.e. this temperature seems to be a good choice in accelerated laboratory tests. After immersion, the tubes were cut into 20-mm wide rings and the concrete filling was removed. Then, split-disk test was conducted on FRP rings to obtain the material properties, such as hoop strength and Young's modulus.

Axial compression test was conducted on a total of 56 SWSSC-filled FRP tubular stub columns after different exposure durations. Two types of cross-section (Fig. 8.2), namely fully filled and doubleskin (FRP as outer tube, stainless steel or FRP as inner tube), were adopted. The measured crosssectional dimensions (outside diameter D and thickness t) of FRP and stainless steel (SS) tubes are listed in Table 8.1, where tube label consists of tube type followed by nominal outside diameter. The height is 400 mm for all columns, except the fully-filled tubes with $D_0=50$ mm, whose height is 150 mm. All the specimens, including unconditioned and conditioned specimens, are listed in Table 8.2, in which the ultimate compressive capacity (N_t) and corresponding unconfined concrete strength (f_c) are also included. The label for stub column consists of outer tube name (tube type followed by nominal diameter), inner tube name (tube type followed by nominal diameter, if applicable), concrete indicator ("C") and aging period ("0" for unconditioned, "1" for 1-month, "3" for 3-month and "6" for 6-month). Taking "G114-S50-C-1" for example, it is a SWSSC-filled double-skin tube with GFRP as outer tube ("G114") and stainless steel as inner tube ("S50"), which is immersed in salt solution for one month. The dimensions of the tube are referred to Table 8.1. In this chapter, if the aging period in specimen label is omitted (e.g., G114-S50-C), it means a group of specimens with the same dimensions but different aging durations.



Table 8.1. Cross-sectiona	l dimensions	of FRP	and SS	tubes
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Tube label	Туре	<i>D</i> (mm)	<i>t</i> (mm)
G50	GFRP	51.1	3.07
G101	GFRP	100.1	3.13
G114	GFRP	115.2	3.13
G165	GFRP	158.2	3.14
C50	CFRP	50.5	2.81
C101	CFRP	99.9	2.81
C114	CFRP	114.6	2.75
C165	CFRP	158.1	2.79
B50	BFRP	50.0	2.71
B101	BFRP	100.0	2.92
B114	BFRP	114.5	2.78
B165	BFRP	157.7	2.71
S50	SS	47.9	2.82
S101	SS	101.8	2.91

Cross-	Unexposed ^a			1-month			3-month			6-month		
section	Specimen	Nt (kN)	fc' (MPa)	Specimen	Nt (kN)	fc' (MPa)	Specimen	Nt (kN)	fc' (MPa)	Specimen	Nt (kN)	fc' (MPa)
	G50-C-0	244	29.8							G50-C-6	274	63.9
	G101-C-0	670	29.8	57/4			N1/A			G101-C-6	842	63.9
	G114-C-0	814	29.8	N/A			N/A			G114-C-6	825	63.9
	G165-C-0	1338	29.8							G165-C-6	1397	63.9
	C50-C-0	388	35.8	C50-C-1	481	39.7	C50-C-3	N/A		C50-C-6	505	66.2
Fully	C101-C-0	1131	35.8	C101-C-1	1363	39.7	C101-C-3	1400 56.5		C101-C-6	1270	66.2
filled	C114-C-0	1416	35.8	C114-C-1	1554	39.7	C114-C-3	1804	56.5	C114-C-6	1712	66.2
	C165-C-0	2372	35.8	C165-C-1	2992	39.7	C165-C-3	3074	56.5	C165-C-6	3012	66.2
	B50-C-0	259	32.8	B50-C-1	255	47.3	B50-C-3	256	51.9	B50-C-6	N/A	
	B101-C-0	656	32.8	B101-C-1	697	47.3	B101-C-3	662	51.9	B101-C-6	666	58.2
	B114-C-0	825	32.8	B114-C-1	837	47.3	B114-C-3	787	51.9	B114-C-6	738	58.2
	B165-C-0	1345	32.8	B165-C-1	1428	47.3	B165-C-3	1396	51.9	B165-C-6	1339	58.2
	G114-S50-C-0	872	32.9							G114-S50-C-6	952	60.0
	G165-S101-C-0	1301	32.9	NT/ A			NT/A			G165-S101-C-6	1234	60.0
	G114-G50-C-0	797	32.9	N/A			N/A			G114-G50-C-6	870	60.0
	G165-G101-C-0	882	32.9							G165-G101-C-6	1005	60.0
	C114-S50-C-0	1375	39.4	C114-S50-C-1	1431	46.3	C114-S50-C-3	1515	48.8	C114-S50-C-6	1537	54.8
Double-	C165-S101-C-0	1698	39.4	C165-S101-C-1	1733	46.3	C165-S101-C-3	1762	48.8	C165-S101-C-6	1728	54.8
skin	C114-C50-C-0	1175	39.4	C114-C50-C-1	1359	46.3	C114-C50-C-3	1360	48.8	C114-C50-C-6	1257	54.8
	C165-C101-C-0	1219	39.4	C165-C101-C-1	1456	46.3	C165-C101-C-3	1301	48.8	C165-C101-C-6	1317	54.8
	B114-S50-C-0	884	32.8	B114-S50-C-1	817	47.3	B114-S50-C-3	783	51.9	B114-S50-C-6	811	58.2
	B165-S101-C-0	1053	32.8	B165-S101-C-1	1048	47.3	B165-S101-C-3	1020	51.9	B165-S101-C-6	1010	58.2
	B114-B50-C-0	651	32.8	B114-B50-C-1	667	47.3	B114-B50-C-3	659	51.9	B114-B50-C-6	690	58.2
	B165-B101-C-0	703	32.8	B165-B101-C-1	789	47.3	B165-B101-C-3	766	51.9	B165-B101-C-6	835	58.2

Table 8.2. Specimen table

a: Adapted from Chapters 3 and 4

8.2.2 Materials

Alkaline-activated slag concrete utilizing seawater and sea sand (SWSSC) was adopted in this study. Composition of the concrete mixture is: slag (360 kg/m³), seawater (190 kg/m³), sea sand (830kg/m³), coarse aggregate (1130 kg/m³), hydrate lime slurry (14.4 kg/m³) and sodium meta-silicate (38.4 kg/m³) as activator. The 3% (percentage weight of slag) sodium meta-silicate activator, which is composed of 47% SiO2 and 36% Na2O, was pre-blended with slag in the dry form before mixing. More details of the mixture, raw material properties and mixing procedures can be found in Chapter 3.

The columns were cast by different batches of concrete with the same mixture. For each batch, six plain SWSSC cylinders were prepared to determine the unconfined concrete strength. After casting, the columns and concrete cylinders were covered by plastic film and stored for 28 days. Triplicate tests on the concrete cylinders were carried for 28-day strength. Then, the columns and the other three cylinders were immersed in 3.5% salt solution at 40 °C. After reaching the required aging duration, the columns and cylinders were tested within five days to ensure that the concrete strength variation is negligible.

E-glass, carbon, and basalt-fiber reinforced polymer (GFRP, CFRP and BFRP) tubes were fabricated by filament winding process with epoxy as matrix. The fibers were oriented in different directions so that the tube offered both hoop and longitudinal strengths. Based on the manufacture data, 20%, 40%, and 40% fibres were in the angle of $\pm 15^{\circ}$, $\pm 40^{\circ}$, and $\pm 75^{\circ}$ with respect to the longitudinal axis of tubes. In this study, only hoop direction properties were measured for both conditioned and unconditioned FRP samples by "split-disk" test (ASTM D2290-16 (2016)). The longitudinal properties (compression and tension) for unconditioned FRP samples were obtained in Chapters 3 and 4, which were used in this study as the FRP tubes were from the same batch. The key material properties of unconditioned FRP tubes are summarized in Table 8.3, in which f_{uh} is hoop strength, f_{ul} is longitudinal compressive strength, E_h is hoop Young's modulus and E_l is longitudinal Young's modulus.

Туре	$f_{\rm uh}$ (MPa)	<i>f</i> _{ul} (MPa)	E _h (GPa)	E_l (GPa))
GFRP	308.8	194.7	25.2	21.5
CFRP	592.8	162.9	24.3	40.0
BFRP	331.1	113.6	23.4	23.9

Table 8.3. Material properties of unconditioned FRP tubes (adapted from Chapters 3 and 4)

SS tubes were made of 316 grade austenitic stainless steel in accordance with AS/NZS 4673 (2001). The stress-strain curve of SS is in round shape with substantial strain hardening. Stainless steel material properties were determined by tensile coupon test and the test results have been reported in Chapters 3 and 4. Since stainless steel tubes were received in two batches, their material properties

were slightly different. The 0.2% proof stress ($f_{0.2}$) for SS tubes used in GFRP double-skin columns was 306.8 MPa for the tube size of 50mmx3mm and 324.4 MPa for the tube size of 101mmx3mm, whilst $f_{0.2}$ for SS tubes in C/BFRP double-skin columns was 228.2 MPa for the tube size of 50mmx3mm and 225.7 MPa for the tube size of 101mmx3mm. Due to the short test duration (up to 6 months), no deterioration was observed in SS tube surface, and it is assumed that its mechanical properties do not degrade due to the exposure.

8.2.3 Experimental setup

The columns and concrete cylinders were aged in water tanks filled with salt solution (Fig. 8.3). The solution temperature was kept as 40 °C using a heating system that consisted of a heating element and a temperature controller. In order to avoid damage of tanks by heavy specimens, the columns were placed in the baskets hanging from steel frames. The water tanks were covered by foam lids and their four sides were insulated by foam panels to reduce heat loss. When the solution level became low due to water evaporation during the immersion test, distilled water was added to ensure that the NaCl concentration remain the same. After exposure, the specimens were taken out, dried naturally and tested.



Fig. 8.3. Conditioning chamber setup

The setup for material property experiments, such as tensile coupon test and "split-disk" test, can be found in Chapter 3. Axial compressive tests on stub columns were conducted on a 5000 kN testing machine with a displacement control and the loading rate is 1.0 mm/min (Fig. 8.4). High strength paste was used to fill the height gap (if any) between tubes and concrete due to shrinkage so that the load can be uniformly applied on both concrete and tubes simultaneously. Three pairs of strain gauges, including one in longitudinal direction and one in hoop direction for each pair, were attached with them located 120 degrees apart at the mid-height of the column. Three LVDTs were applied to measure the axial deformation of the column along the whole height. The test data (e.g., load, deflection and strain), were measured and recorded by a data acquisition system.



Fig. 8.4. Compressive test setup and instrumentation

8.3 Results and discussions

8.3.1 Seawater and sea sand concrete (SWSSC)

In order to understand the effect of aging time on the strength (f_c ') of SWSSC alone, the strength of concrete cylinders at 28-day and at the date of columns test (after exposure) is measured in according to AS 1012.9 (2014). The ratio of f_c ' at test date and at 28-day is plotted in Fig. 8.5 against the aging time. Considerable increase in concrete strength is observed during the early period of immersing in salt solution at 40°C. Such increase in strength is attributed to a further polymerization of cementitious material (slag) in concrete (Chapter 2). After 3 months, the concrete strength keeps almost stable indicating that the salt solution does not have a detrimental effect on SWSSC. Therefore, the effect of concrete strength increase should be excluded when assessing the durability performance of stub columns.



8.3.2 FRPs

The capacity change of SWSSC-filled FRP tube is a result of the strength change of concrete and FRP. Section 8.3.1 concludes that SWSSC strength does not reduce after exposure to salt solution. The property degradation of FRP alone should be first examined before assessing the durability

performance of stub columns. "Split-disk" test was conducted on conditioned and unconditioned FRP rings (three identical samples for each case), and the stress-strain curves were obtained, from which the hoop strength (f_{uh}) and Young's modulus in hoop direction (E_h) were determined (ASTM D2290 2016).

The stress-strain curve overall displays a linear shape for both unconditioned and conditioned samples. The aging does not affect the shapes of the curves. The exposed-to-unexposed ratio of f_{uh} and E_h are plotted versus the aging time as shown in Fig. 8.6, in which the error bar stands for standard deviation. CFRP shows the best and BFRP shows the worst durability performance in terms of hoop strength (Fig. 8.6a). The hoop strength reduction after 6-month exposure is about 8%, 23% and 39% for CFRP, GFRP and BFRP respectively. On the other hand, the Young's modulus degradation is insignificant for all FRPs, which is in agreement with the reported study (Wang et al. 2017).



8.3.3 SWSSC-filled stub columns

8.3.3.1 Load-axial strain and hoop-axial strain curves

The failure mode for stub columns is FRP tube rupture in hoop direction, except for specimen groups of C165-C101-C, B114-B50-C and B165-B101-C, whose concrete could not be effectively confined due to the damage of inner FRP tubes. The typical failure modes for some ruptured specimens (unexposed and 6-month exposed) are shown in Fig. 8.7. After exposure, the failure mode does not change but the rupture becomes less explosive which is caused by the property degradation of FRP tube. It is found that the colour of GFRP surface changes from bright green to brown green after aging, which is likely to be caused by the chemical composition change (such as "leaching" and "etching", Chen 2007) of FRP. The colour change of CFRP and BFRP is not obvious. During the test, the buckling of FRP tube occurred very early accompanied by a slight drop of applied load. However, the stub columns could still increasingly sustain the load until tube rupture.



Fig. 8.7. Typical failure modes of unexposed and exposed specimens

The load-axial strain curves of all columns are shown in Fig. 8.8, in which the axial strain is equal to the overall axial shortening (measured by LVDTs) divided by column height. Most of the curves display a bilinear shape, which is in agreement with the shape of FRP wrap-confined concrete (Ozbakkaloglu et al. 2013). Because of the buckling of FRP tube, slightly load drop is shown in the load-axial strain curves. In Fig. 8.8, the unconfined concrete capacity is highlighted by circles. The ultimate capacity of SWSSC-filled FRP tube is much higher than the unconfined concrete capacity, indicating a great enhancement of concrete strength and ductility caused by confinement effect. The ultimate capacity of exposed specimen is higher than that of unexposed specimen as the unconfined concrete strength increases after aging. Therefore, a direct comparison of ultimate capacity is not helpful in assessing the durability performance of columns. After aging, the shape of load-axial strain curves for most specimens do not change. However, the second region (strain hardening part) of loadaxial strain curves of some conditioned specimens with small confinement (e.g., G114-C-6, G165-C-6, B114-C-6, B165-C-6, G165-S101-C-6 and B165-S101-C-6) becomes flat. For double-skin specimens without tube rupture, a descending part of the curve is observed. In this case, the inward expansion of concrete is not effectively restrained due to the damage of FRP inner tube and the outer tube cannot reach its ultimate condition. As shown in Fig. 8.8, the ultimate axial strain decreases after exposure since the hoop strength of FRP tube is reduced. It is also found that the initial stiffness of stub columns increases after aging. The reason is the increase of elastic modulus of concrete due to the strength development.







Fig. 8.8. Load-axial strain curves and hoop-axial strain curves for SWSSC-filled FRP tubes

The hoop-axial strain curves are also shown in Fig. 8.8, where the hoop strain is obtained from average strain gauge readings (tension as negative). In some specimens (e.g., G114-C-0), the strain gauges were damaged by tube buckling and the curve is terminated before reaching ultimate state. Generally, the aging does not change the shape of hoop-axial strain curves. For a given axial strain, the hoop strain (absolute value) of exposed specimens is higher than that of corresponding unexposed specimens. It is mainly caused by the reduction of confining pressure and the increase of unconfined concrete strength.

8.3.3.2 Strength

The confined concrete strength (f_{cc} ') is equal to the load carried by concrete divided by its crosssectional area. At the ultimate state, multiple buckles have occurred in FRP tube and it is assumed the load carried by FRP tube is zero. The load carried by SS tube is assumed to be approximately equal to its yield capacity (i.e., products of $f_{0.2}$ and cross-sectional area). As discussed in Section 8.3.1, concrete strength increased after aging. In order to avoid the influence of f_c ' increase, normalized confined strength (f_{cc} '/ f_c ') is adopted herein to assess the environmental effects on column strength.

Table 8.4 summarizes the f_{cc}'/f_{c}' data for unexposed specimens and the ratio of f_{cc}'/f_{c}' of exposed to unexposed specimens. As expected, with the increase of tube diameter, f_{cc}'/f_{c}' increases due to the decrease in confining pressure. The average values for each FRP type are plotted in Fig. 8.9 with error bars standing for standard deviations. Obvious strength degradation for SWSSC-filled FRP tubes is observed after a certain time of aging. CFRP shows a superior performance than GFRP and BFRP, which agrees with the durability performance of FRP alone discussed in Section 8.3.2. The normalized strength retentions after 6-month aging are similar for GFRP and BFRP columns. Furthermore, the durability performance of double-skin tubes is similar to that of fully filled tubes as the confined concrete strength is mainly related to the hoop strength of outer FRP tube. Generally speaking, the strength loss is about 45%, 28%, and 44% for GFRP, CFRP and BFRP columns subjected to the salt solution for 6 months at 40 °C. As shown in Table 8.4, there is no clear trend of the effect of tube size, which determines confining stiffness, on the normalized strength loss with the aging time. It seems that the strength degradation for SWSSC-filled GFRP and BFRP tubes subjected to salt solution at 40 °C is considerable, which is not as superior as expected. Special attention should be paid in their applications at high temperatures.

Spacimon	Unexposed	Exposed-to-	unexposed ra	tio of f_{cc}'/f_c'	Spaaiman	Unexposed	Exposed-to-unexposed ratio of $f_{\rm cc}'/f_{\rm c}'$			
specifien	$f_{\rm cc}'/f_{\rm c}'$	1-month	3-month	6-month	Specifici	$f_{\rm cc}'/f_{\rm c}'$	1-month	3-month	6-month	
G50-C	5.16			0.52	G114-S50-C	3.02			0.61	
G101-C	3.25	NI/A		0.59	G165-S101-C	3.04	NI/A		0.51	
G114-C	2.93	N/A		0.47	G114-G50-C	3.34	N/A		0.60	
G165-C	2.48			0.49	G165-G101-C	2.61			0.63	
C50-C	6.86	1.12	N/A	0.70	C114-S50-C	4.41	0.89	0.90	0.81	
C101-C	4.52	1.09	0.78	0.61	C165-S101-C	3.77	0.87	0.84	0.73	
C114-C	4.24	0.99	0.81	0.65	С114-С50-С	4.06	0.98	0.93	0.77	
C165-C	3.63	1.14	0.82	0.69	C165-C101-C	2.97	1.02	0.86	0.78	
В50-С	5.06	0.68	0.62	N/A	В114-S50-С	3.26	0.63	0.55	0.51	
B101-C	2.87	0.74	0.64	0.57	B165-S101-C	2.60	0.69	0.61	0.53	
B114-C	2.70	0.70	0.60	0.50	В114-В50-С	2.70	0.71	0.64	0.60	
B165-C	2.26	0.74	0.66	0.56	B165-B101-C	2.08	0.78	0.69	0.67	

Table 8.4. Ratio of confined concrete strength-to-unconfined concrete strength



8.3.3.3 Ultimate axial strain

The ultimate axial strain (ε_{cu}) is defined as the axial strain corresponding to outer FRP tube rupture or corresponding to the peak load if the tube is not ruptured. Table 8.5 and Fig. 8.10 summarize the ratio of ultimate axial strain of exposed specimens to that of the unexposed. Ultimate axial strain of unexposed fully filled tubes decreases with increase in the tube dimeter as the confining pressure decreases. However, the decreasing trend for double-skin tubes is not clear since the void ratio $(D_i/(D_o-2t_o))$ also affects the ultimate axial strain. After exposure, significant ultimate axial strain reduction was observed for both fully filled and double-skin tubes (Fig. 8.10). This is mainly caused by the reduction of FRP hoop strength and Young's modulus. In terms of ultimate axial strain, CFRP shows the best and BFRP shows the worst durability performance. The effect of tube size on ε_{cu} reduction is not obvious based on the data shown in Table 8.5.



Table 8.5. Ultimate axial strain

Specimon	Unexposed	Exposed-t	o-unexposed	l ratio of ε_{cu}	Spacimon	Unexposed	Exposed-t	o-unexposed	ratio of ε_{cu}
specifien	Ecu	1-month	3-month	6-month	Specifien	Ecu	1-month	3-month	6-month
G50-C	0.027			0.72	G114-S50-C	0.033			0.66
G101-C	0.036	NT/A		0.74	G165-S101-C	0.036	NT/A		0.71
G114-C	0.031	N/A		0.38	G114-G50-C	0.033	N/A		0.41
G165-C	0.028			0.57	G165-G101-C	0.032			0.57
C50-C	0.063	0.95	N/A	0.99	C114-S50-C	0.052	0.77	N/A	0.79
C101-C	0.052	0.85	0.78	0.62	C165-S101-C	0.058	0.78	0.69	0.64
C114-C	0.051	0.76	0.83	0.86	С114-С50-С	0.056	0.91	0.89	0.64
C165-C	0.046	0.97	0.80	0.78	C165-C101-C	0.028	1.03	0.93	0.58
В50-С	0.053	0.71	0.74	N/A	В114-S50-С	0.040	0.52	0.46	N/A
B101-C	0.041	0.69	0.53	0.37	B165-S101-C	0.035	0.60	0.61	0.39
B114-C	0.032	0.66	0.48	0.39	В114-В50-С	0.034	0.66	0.61	0.28
B165-C	0.034	0.59	0.51	0.43	B165-B101-C	0.024	0.48	0.22	0.24

8.3.3.4 Rupture stress of FRP tube

Rupture stress (σ_{uh}) of columns failed by tube rupture is discussed in this Section. The rupture stress cannot be directly measured from the experiments and it is "back-calculated" from experimental strength of stub columns (i.e., confined concrete strength f_{cc} '). Based on existing theoretical study on concrete fully filled FRP tubes/wraps (Teng et al. 2007; Chapter 6), the relationship between f_c ' and f_{uh} is:

$$f_{cc}^{'} = f_{c}^{'} + 3.5f_{l} \tag{8.1}$$

$$f_l = \frac{\sigma_{uh} t_o}{\left(D_o - 2t_o\right)/2} \tag{8.2}$$

where f_l is confining pressure. Eqs. (8.1-2) are also applied to double-skin tubes by assuming the annual concrete is uniformly confined, which was validated by Yu et al. (2010). The calculated σ_{uh} for unexposed specimens and exposed-to-unexposed ratio of σ_{uh} are summarized in Table 8.6.

Hoop strength (f_{uh}) of unexposed FRP tube obtained from material test (Section 8.2.2) is 308.8, 592.8 and 331.1 MPa for GFRP, CFRP and BFRP respectively. As shown in Table 8.6 the rupture stress of unexposed SWSSC fully filled G/BFRP tubes is in good agreement with the hoop strength. The rupture stress of some unexposed CFRP columns (e.g., C114-C-0 and C165-C-0) is higher than f_{uh} probably due to: (1) bending effect in "split-disk" test that causes an under-estimation of f_{uh} and it is more obvious for CFRP that possesses higher modulus of elasticity; (2) FRP tubes were cut from different long tubes and property variation may exist between these tubes. Similar observation was reported in the Chapter 6 using the same materials. The rupture stress in unexposed double-skin tubes is higher than f_{uh} . It is mainly caused by the underestimation of the contribution of inner SS or FRP tubes. However, this study mainly focuses on the durability performance by assessing the relative strength degradation of stub columns after aging. The exposed-to-unexposed ratio of rupture stress is the major concern instead of its absolute value.

Spaaiman	Unexposed	Exposed-to	o-unexposed	ratio of $\sigma_{\rm uh}$	Spacimon	Unexposed	Exposed-to-unexposed ratio of σ_{uh}			
Specifien	$\sigma_{\rm uh}$ (MPa)	1-month	3-month	6-month	specifien	$\sigma_{\rm uh}$ (MPa)	1-month	3-month	6-month	
G50-C	259			0.88	G114-S50-C	357			0.75	
G101-C	288	N/A		0.86	G165-S101-C	502	N/A		0.50	
G114-C	286			0.43	G114-G50-C	409	N/A		0.78	
G165-C	304			0.30	G165-G101-C	368			0.72	
C50-C	478	1.26	N/A	1.21	C114-S50-C	761	1.01	1.07	1.05	
C101-C	604	1.23	1.14	0.92	C165-S101-C	853	0.97	0.97	0.89	
C114-C	656	1.09	1.18	1.01	C114-C50-C	684	1.15	1.13	0.96	
C165-C	735	1.32	1.19	1.05	C165-C101-C ^a	N/A				
B50-C	314	0.87	0.84	N/A	В114-S50-С	415	0.68	0.56	0.52	
B101-C	283	0.86	0.70	0.61	B165-S101-C	422	0.72	0.57	0.43	
B114-C	312	0.76	0.58	0.38	B114-B50-C ^a	NI/A				
B165-C	330	0.76	0.60	0.37	B165-B101-C ^a	1N/A				

Table 8.6. Rupture stress of FRP tube in stub columns

Exposed-to-unexposed ratio of σ_{uh} is summarized in Table 8.6 and plotted in Fig. 8.11, where the hoop strength (f_{uh}) retention data of FRP obtained from "split-disk" test (Section 8.2.2) are also given for comparison. As expected, both σ_{uh} and f_{uh} decrease with aging time, and SWSSC-filled CFRP tubes have higher strength retention than SWSSC-filled GFRP and BFRP tubes. It is found that the rupture stress retentions in GFRP and BFRP tubes (denoted as "column" in Fig. 8.11) are lower than the corresponding hoop strengths (denoted as "material" in Fig. 8.11). For CFRP columns, no reduction in the exposed-to-unexposed ratio of σ_{uh} is found after aging, which is probably due to the

fact that the reduction in hoop strength of CFRP is very small after aging. It seems that the rupture stress retentions in GFRP and BFRP columns are lower than corresponding retentions in material hoop strength, especially for columns with low confinement. This finding will be further discussed in Section 8.4.3 and will be incorporated into modified load-axial strain model.



8.3.3.5 Rupture strain of FRP tube

Rupture strain (ε_{hu}) of FRP tube in stub column is determined by hoop strain gauges. The results, including rupture strain of unexposed specimens and exposed-to-unexposed ratio of ε_{hu} , are summarized in Table 8.7 and Fig. 8.12. The ratio of ε_{hu} data are rather scattered in this study because the strain measured by strain gauges is "local" and the buckling of FRP tube reduces the measurement accuracy. Generally, the rupture strain reduction is slower than rupture stress reduction since Young's modulus of FRP tube only decreases slightly as discussed in Section 8.2.2. This result is similar to the finding in Xie et al. (2018). It is necessary to mention that ACI code (ACI440.2R-08) regulates that the reduction factor for rupture strain of aged FRP is the same as that for rupture stress. On the other hand, Szenz and Pantelides (2006) found that the rupture strain reduction caused by aging was less than 5%.



Specimon	Unexposed	Exposed-to	o-unexposed	ratio of ε_{hu}	Spaciman	Unexposed	Exposed-to	Exposed-to-unexposed ratio of ε_{hu}			
Specifien	$\mathcal{E}_{\mathrm{hu}}$	1-month	3-month	6-month	Specifien	$\mathcal{E}_{\mathrm{hu}}$	1-month	3-month	6-month		
G50-C	-0.007			1.07	G114-S50-C	-0.011			1.12		
G101-C	-0.013	N/A		1.34	G165-S101-C	-0.014	N/A		0.84		
G114-C	N/A			N/A	G114-G50-C	-0.012	N/A	0.74			
G165-C	-0.013			1.06	G165-G101-C	-0.006			1.19		
C50-C	N/A				C114-S50-C	-0.014	0.73	1.22	1.19		
C101-C	-0.011	1.42	1.49	0.96	C165-S101-C	-0.015	0.79	0.67	0.77		
C114-C	-0.013	0.88	1.14	1.41	С114-С50-С	-0.014	1.27	0.96	0.86		
C165-C	-0.012	1.39	1.32	1.28	C165-C101-C	N/A					
В50-С	N/A				B114-S50-C	-0.016	0.73	0.51	0.57		
B101-C	-0.014	1.29	0.74	1.26	B165-S101-C	-0.016	0.62	0.65	0.58		
B114-C	-0.014	1.09 0.66 0		0.77	В114-В50-С	NT/A					
B165-C	-0.018	0.99	0.92	0.72	B165-B101-C	1N/A					

Table 8.7. Rupture strain of FRP tube in stub columns

8.3.3.6 Volume strain

Volume strain of concrete is defined as:

$$\varepsilon_{v} = \varepsilon_{c,SG} + 2\varepsilon_{h} \tag{8.3}$$

where $\varepsilon_{c,SG}$ is axial strain obtained from strain gauges (positive), ε_h is hoop strain (negative). A positive value of ε_v means concrete volume contraction and a negative stands for expansion. The volume strain-axial strain (obtained by strain gauges) relationships for specimen group B114-C are plotted in Fig. 8.13. Imran and Pantazopoulou (1996) found that the axial strain at zero volume strain (ε_{c0}) marks the onset of strength degradation of unconfined concrete. These points are identified by circles in Fig. 8.13 and corresponding axial strains are also given. When the influence of concrete strength on ε_{c0} is excluded, it is found that the exposure does not affect the unconfined concrete property (i.e., dilation and strength) obviously.



Fig. 8.13. Volume-axial strain relationship (B114-C)

8.3.4 Degradation mechanism

As discussed previously, the property of plain concrete does not deteriorate after exposure to salt solution. The strength degradation of stub columns is mainly caused by the deterioration of FRP material. Moisture absorption and diffusion of chemicals (e.g., alkali) can induce delamination, swelling, cracking and plasticization of FRP matrix. Glass fiber degradation involves the process of "leaching", which is the diffusion of alkali ions out of glass structure, and "etching", in which the hydroxyl ions break the Si-O-Si (Chen 2007). Carbon fiber is inert to chemical exposure, and hence, shows the best durability. The chemical composition of basalt fiber is similar to glass fiber (except the inclusion of iron oxide in the former Parnas et al. 2007), and its degradation mechanism is similar to glass fiber. The interphase between fiber and matrix plays an important role in load transfer. The degradation mechanism for interphase includes matrix osmotic cracking, interfacial debonding and delamination (Bradshaw et al. 1997).

The strength degradation of stub columns is mainly caused by the reduction of confining pressure, which is related to FRP hoop strength. The reduction of Young's modulus of FRP leads to the confining stiffness decrease. Based on Teng et al's study (2009), if the confining stiffness ratio is lower than a limit, the concrete cannot be effectively confined. Furthermore, due to the existence of longitudinal fibers, FRP deterioration could cause a reduction of the load carried by FRP tube.

8.4 Predictions

8.4.1 Prediction for load-axial strain curves of fully filled tubes

8.4.1.1 Dilation property

Chapter 6 proposed an analysis-oriented load-axial strain model for SWSSC fully filled FRP tube, in which FRP tube offers both longitudinal and hoop strength. Before tube buckling, the hoop-axial strain relationship (i.e., dilation model) is:

$$\varepsilon_c = \frac{(\Phi - \Phi K \varepsilon_h) \varepsilon_{co}}{1 - \Phi K k_l v_l \varepsilon_{co}}$$
(8.4)

$$\Phi = 0.85 \left\{ \left[1 + 0.75(\frac{-\varepsilon_h}{\varepsilon_{co}}) \right]^{0.7} - \exp[-7(\frac{-\varepsilon_h}{\varepsilon_{co}})] \right\}$$
(8.5)

$$K = \frac{8}{f_c} \cdot \frac{E_h t_o}{(D_o - 2t_o) / 2 \cdot (\nu_l \nu_h - 1)}$$
(8.6)

where ε_{co} is axial strain of unconfined concrete at f_c ', which is $0.000937 \sqrt[4]{f_c}$ as suggested by Popovic (1973), k_1 is a factor taken as 0.8, v_l and v_h are Poisson's ratios in longitudinal and hoop directions.

After tube buckling, the dilation model for concrete-filled FRP wrap (Teng et al. 2007) is applied. More details, such as buckling prediction, of the dilation model can be found in Chapter 6. Fig. 8.14 plots some typical hoop-axial strain curves, including both experimental curves and predicted curves, of unexposed and exposed specimens. During calculation, the experimental properties (e.g., f_{uh} , E_h) of aged FRP (Section 8.3.2) were used. Generally, the dilation model proposed in Chapter 6 provides similar accuracy in predicting hoop-axial strain relationship of exposed and unexposed specimens. Therefore, this dilation model can be applied to exposed specimens without modification.



Fig. 8.14. Prediction for hoop-axial strain relationship of fully filled tubes without or with 6-month exposure

8.4.1.2 Load-axial strain curves

After determining the hoop-axial strain relationship, the analysis-oriented load-axial strain curve can be developed by numerical iterative process. The load-axial strain curve includes two parts: (1) before tube buckling, the biaxial stress state in FRP tube, Poisson's effect and load carried by FRP tube are considered and (2) after tube buckling, the existing model for concrete-filled FRP wrap is adopted except accounting for the residual strength of FRP tube. More information for generating the loadaxial strain curves of SWSSC-filled FRP tubes can be found in Chapter 6. The experimental material properties of aged FRP were adopted to predict the load-axial strain curves of conditioned columns. A comparison of the experimental and predicted load-axial strain curves for some unconditioned and conditioned specimens is shown in Fig. 8.15.



Fig. 8.15. Prediction for load-axial strain curves of fully filled tubes without or with 6-month exposure

The predicted load-axial strain curves are in good agreement with experimental curves for unexposed specimens, which has been validated in Chapter 6. The accuracy of the prediction for aged specimens with high confinement stiffness (e.g., G101-C-6, C101-C-6 and B101-C-6) is acceptable. However, obvious over-estimation is observed for aged specimens with low confinement stiffness (e.g., G165-C-6 and B165-C-6). As the hoop-axial strain relationship prediction is reasonable (Fig. 8.14), the reason for the over-estimation should be the concrete cannot be effectively confined at low confinement stiffness. Some modifications will be made to consider this phenomena in the following discussion. It should be noted that the predicted ultimate capacity and strain for specimen group C165-C is much lower than experimental values but the shapes of the predicted curves are in good agreement. This is caused by the fact that hoop strength of CFRP rings used for "split-disk" test is probably lower than that of CFRP tubes used in stub columns.

8.4.1.3 Modification of Li et al.'s method (2018b)

Confinement stiffness ratio ($\rho_{\rm K}$) is commonly used in literatures (Teng et al. 2009) to represent the confining stiffness and its definition is:

$$\rho_{\kappa} = \frac{2E_{h}t_{o}}{(f_{c}^{\prime} / \varepsilon_{co})(D_{o} - 2t_{o})}$$

$$(8.7)$$

Based on Teng et al.'s (2009) study, if $\rho_{\rm K}$ is less than 0.01, it is assumed that $f_{\rm cc}'=f_{\rm c}'$, which means confinement effect is ignored, otherwise, the concrete is effectively confined. The confinement stiffness ratios of SWSSC fully filled FRP tubes are listed in Table 8.8 and all the values of $\rho_{\rm K}$ is larger than 0.01.

Specimen	Unexposed	1-month	3-month	6-month
G50-C	0.231			0.100
G101-C	0.113	NT/A		<u>0.049</u>
G114-C	0.097	1N/A		0.042
G165-C	0.070			<u>0.030</u>
C50-C	0.467	0.391	0.278	0.222
C101-C	0.222	0.186	0.132	0.106
C114-C	0.188	0.157	0.112	0.089
C165-C	0.136	0.114	0.081	0.065
В50-С	0.180	0.119	0.104	0.091
B101-C	0.092	0.061	0.053	0.046
B114-C	0.076	0.050	0.044	<u>0.038</u>
B165-C	0.053	0.035	0.031	0.027

Table 8.8. Confinement stiffness ratio of SWSSC fully filled FRP tubes

^a: Number with underline means $\rho_{\rm K} < 0.05$

However, the experimental load-axial strain curves (Fig. 8.8) indicate that the confinement in some aged specimens is not fully effective. After comparing all the predicted and experimental curves (part of the comparison is shown in Fig. 8.15), it is found that if $\rho_{\rm K}$ is less than 0.05, the confinement becomes partly effective. In this study, a confinement effectiveness factor ($k_{\rm ce}$) is proposed in determining the confined concrete strength as shown in Eq. (8.8):

$$f_{cc}' = f_c' + 3.5k_{ce}f_l \tag{8.8}$$

The relationship between experimental k_{ce} and ρ_{K} is shown in Fig. 8.16, in which the assumption in Teng's method is also plotted by dash-dotted line. Based on Fig. 8.16, it is proposed that if ρ_{K} is larger than 0.01 but less than 0.05, the confinement is partly effective and k_{ce} can be determined by:

$$k_{ce} = \begin{cases} 0, & \rho_{K} < 0.01 \\ 25\rho_{K} - 0.25, & 0.01 \le \rho_{K} \le 0.05 \\ 1, & \rho_{K} > 0.05 \end{cases}$$
(8.9)

It should be pointed out that Eq. (9) is obtained based on the test results in this investigation which covers three types of FRP tubes and SWSSC. Further research may be needed to extend the validity range of Eq. (9). Therefore, the initial equation to calculate f_{cc} ' in Li et al.'s method (2018b) (Eq. (8.1)) is replaced by Eq. (8.8) and the predicted load-axial strain curves after modification is shown in Fig. 8.15 (dash-dotted line denoted as "Pre - modified"). Comparing to the original method in Chapter 6, the slope of the second part of "modified" predicted curves becomes closer to that of experimental curves and the predicted ultimate capacity is in better agreement with experimental capacity.



Fig. 8.16. Relationship between confining effectiveness factor and confinement stiffness ratio

8.4.2 Ultimate capacity prediction

At the ultimate state, the load carried by FRP outer or inner tube is ignored since FRP tube is seriously buckled. It is assumed that the load carried by stainless steel inner tube is equal to its yield capacity,

which is equal to the product of cross-sectional area and 0.2% proof stress ($f_{0.2}$). The confined concrete strength is determined by Eq. (8.8) for both solid circular concrete in fully filled tube and annual concrete in double-skin tube. It is necessary to mention that some double-skin tubes did not fail by tube rupture but Eq. (8.8) is still used for consistency. The material properties of FRP are obtained from "split-disk" test on unconditioned and aged FRP rings (Section 8.3.2). It is assumed that $f_{0.2}$ of stainless steel does not change after exposure.

The prediction for ultimate capacity of SWSSC-filled fully-filled and double-skin FRP tubes is summarized in Tables 8.9-8.10 and Fig. 8.17, in which the experimental capacity and the capacity ratio of prediction-to-experiment are also presented. The prediction accuracy is overall acceptable with the average ratio ranges from 0.86 to 1.04. The prediction for SWSSC-filled CFRP tubes is much conservative and the explanation has been given in Section 8.3.3.4 and 8.4.1.2. As shown in Table 8.10 and Fig. 8.17b, the predicted capacities for double-skin tubes (SS as inner tube) are lower than experimental capacities probably due to the under-estimation of loads carried by SS tubes. In conclusion, the capacity of aged columns can be accurately predicted if adopting the material property of aged samples and considering the environmental effect on confinement effectiveness (Eqs. (8.8-9)).



Caralina	Experiment	(kN)		U U	Prediction (kN)			Ratio of prediction-to-experiment			
specimen	unexposed	1-month	3-month	6-month	unexposed	1-month	3-month	6-month	unexposed	1-month	3-month	6-month
G50-C	244			274	282			281	1.16			1.03
G101-C	670	NI/A		842	705	NI/A		815	1.05	NT/A		0.97
G114-C	814	IN/A		825	856	N/A	N/A	953	1.05	IN/A		1.15
G165-C	1338			1397	1350			1474	1.01			1.05
С50-С	388	481	N/A	505	467	463	479	481	1.20	0.96	N/A	0.95
C101-C	1131	1363	1400	1270	1114	1121	1215	1255	0.99	0.82	0.87	0.99
C114-C	1416	1554	1804	1712	1312	1325	1456	1515	0.93	0.85	0.81	0.89
C165-C	2372	2992	3074	3012	2040	2079	2348	2481	0.86	0.69	0.76	0.82
В50-С	259	255	256	N/A	271	274	241	226	1.04	1.07	0.94	N/A
B101-C	656	697	662	666	729	784	725	684	1.11	1.12	1.10	1.03
B114-C	825	837	787	738	857	942	822	780	1.04	1.12	1.04	1.06
B165-C	1345	1428	1396	1339	1347	1286	1224	1250	1.00	0.90	0.88	0.93
Mean									1.04	0.94	0.91	0.99
COV ^a									0.09	0.15	0.12	0.09

Table 8.9. Capacity prediction for SWSSC fully filled tubes

^a: COV=coefficient of variation

Spacimon	Experiment	(kN)			Prediction (kN)			Ratio of prediction-to-experiment			
specimen	unexposed	1-month	3-month	6-month	unexposed	1-month	3-month	6-month	unexposed	1-month	3-month	6-month
G114-S50-C	872			952	804			838	0.92			0.88
G165-S101-C	1301	NI/A		1234	1041	NI/A		1056	0.80	NI/A		0.86
G114-G50-C	797	N/A		870	660	1N/A		693	0.83	1N/A		0.80
G165-G101-C	882			1005	794			810	0.90			0.81
С114-S50-С	1375	1431	1515	1537	1158	1190	1188	1207	0.84	0.83	0.78	0.79
C165-S101-C	1698	1733	1762	1728	1361	1413	1418	1454	0.80	0.82	0.80	0.84
C114-C50-C	1175	1359	1360	1257	1057	1090	1088	1107	0.90	0.80	0.80	0.88
C165-C101-C	1219	1456	1301	1317	1201	1255	1260	1297	0.99	0.86	0.97	0.98
В114-S50-С	884	817	783	811	775	842	748	715	0.88	1.03	0.96	0.88
B165-S101-C	1053	1048	1020	1010	939	906	872	886	0.89	0.86	0.85	0.88
B114-B50-C	651	667	659	690	676	743	649	616	1.04	1.11	0.98	0.89
B165-B101-C	703	789	766	835	765	731	696	710	1.09	0.93	0.91	0.85
Mean									0.91	0.91	0.88	0.86
COV ^a									0.10	0.12	0.09	0.06

Table 8.10. Capacity prediction for SWSSC-filled double-skin tubes

^a: COV=coefficient of variation

8.5 Comparison with ACI 440.2R-08

ACI design guideline (ACI 440.2R-08 2008, "Guide for the design and construction of externally bonded FRP systems for strengthening concrete structures") for FRP systems considers the environmental effects on degradation of mechanical properties by introducing an environment reduction factor (C_E). It specifies that C_E for FRP system with exterior exposure (bridges, piers, and unenclosed parking garages) is 0.85 and 0.65 for CFRP and GFRP respectively. Some other standards, such as CAN/CSA-S6-00 (2005), adopted the same concept in considering the environment effect. The tensile strength of FRP after aging can be estimated by:

$$f_{fu} = C_E f_{fu}^*$$
 (8.10)

and the rupture strain can be estimated by:

$$\varepsilon_{fu} = C_E \varepsilon_{fu}^* \tag{8.11}$$

The modulus of elasticity is not affected by environmental conditions.

The experimental C_E can be calculated if knowing the rupture stress of FRP in aged (6-month exposure, corresponding to f_{fu}) and unaged (corresponding to f_{fu}^*) columns. Similar to the calculation in Section 8.3.3.4 except using Eq. (8.8) to replace Eq. (8.1), the experimental C_E is plotted in Fig. 8.18, in which the specified value in ACI 440.2R-08 (2008) is also presented by dash lines. Since C_E is "back-calculated" from confined concrete strength with some assumptions, the values are scattering. The current guideline does not cover BFRP system. As shown in Fig. 8.18(a) and (c), experimental values of C_E for GFRP and BFRP are similar in this study. A lower bound C_E value of 0.5 is suggested for GFRP and BFRP (as dash-dot line in Fig. 8.18). This value is slightly lower than 0.65 in ACI 440.2R-08 (2008), which reflects to some extent more severe condition with temperature effects. The reduction factor for CFRP can still be taken as 0.85.





8.6 Conclusions

This chapter presents a durability study on seawater and sea sand concrete (SWSSC), FRP, and SWSSC-filled G/C/B-FRP tubular stub columns exposed to an artificial seawater at 40 °C for durations up to 6 months. The following conclusions can be made.

(1) Strength of seawater and sea sand concrete is not reduced after exposure. However, considerable hoop strength reduction occurs in GFRP and BFRP (23% and 39% loss respectively after 6 months) whereas the reduction for CFRP is only 8%. Environmental effects on Young's modulus of FRP is insignificant.

(2) Normalized strength (i.e., f_{cc}'/f_c') and ultimate strain of stub columns decrease gradually with the increase of exposure duration. Durability performances of fully filled and double-skin tubes are similar and SWSSC-filled CFRP tubes perform superior than SWSSC-filled G/BFRP tubes in terms of residual strength.

(3) Rupture stress, which is "back-calculated" from experimental compressive strength, of FRP tube in columns subjected to environment aging decreases faster than hoop strength degradation of FRP alone.

(4) Strength degradation of stub columns is mainly caused by the deterioration of FRP material. Environmental aging leads to a partly effective confinement for columns with low confining stiffness. A confinement effectiveness factor (k_{ce}) is proposed in developing the load-axial strain curves to consider the environmental effects on confinement effectiveness.

(5) Based on the method proposed in Chapter 6 with appropriate modification and experimental material properties of conditioned FRP samples, the load-axial strain curves of fully-filled tubes are predicted, and the predictions are in good agreement with experimental curves. The predicted compressive capacities of SWSSC-filled tubes (both fully filled and double-skin) agree well to experimental data indicating the method suggested in this study to be reasonable.

(6) The testing data in this study confirmed that environmental reduction factor specified in design guideline (e.g. ACI 440.2R-08) for CFRP (i.e. 0.85) generally provides a safe estimation on the strength reduction for FRP tubes within SWSSC environment. For SWSSC-filled GFRP and BFRP tubes, the testing data in this study indicated that a slightly smaller reduction factor of 0.5 instead of 0.65 may be adopted for environment with a temperature of 40 degrees. The long-term performance of hybrid construction is outside the scope of this paper although future research is being planned to correlate accelerated corrosion test results with long-term performance.

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References

ACI 440.2R-08 (2008), Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures, American Concrete Institute, Farmington Hills, MI.

AS1012.9:2014 (2014), Methods of testing concrete - Compressive strength tests - Concrete, mortar and grout specimens, Standards Australia, Sydney.

AS/NZS 4673:2001 (2001), Cold-formed stainless steel structures, Standards Australia, Sydney.

ASTM D1141-98 (2013), Standard practice for the preparation of substitute ocean water, American Society for Testing and Materials, West Conshohocken, PA.

ASTM D2290-16 (2016), Standard Test Method for Apparent Hoop Tensile Strength of Plastic or Reinforced Plastic Pipe, American Society for Testing and Materials, West Conshohocken, PA.

Bank, L.C. (2006), Composites for construction: Structural design with FRP materials, John Wiley & Sons, Hoboken, New Jersey.

Boumarafi, A., Abouzied, A. and Masmoudi, R. (2015), Harsh environments effects on the axial behaviour of circular concrete-filled fibre reinforced-polymer (FRP) tubes, Composites Part B: Engineering, 83, 81-7.

Bradshaw, R.D. and Brinson, L.C. (1997), Physical aging in polymers and polymer composites: An analysis and method for time-aging time superposition, Polymer Engineering and Science, 37, 31-44.

CAN/CSA-S6-00(R2005), Canadian Highway Bridge Design Code.

Chen, G.M., He, Z.B., Jiang, T., Chen, J.F. and Teng JG (2017a), Axial compression tests on FRPconfined seawater/sea-sand concrete, 6th Asia-Pacific Conference on FRP in Structures.

Chen, G.M., He, Z.B., Jiang, T., Chen, J.F. and Teng JG (2017a), Effects of seawater and sea-sand on the behaviour of FRP-confined concrete, 6th Asia-Pacific Conference on FRP in Structures.

Chen, Y., Davalos, J.F., Ray, I. and Kim, H.-Y. (2007), Accelerated aging tests for evaluations of durability performance of FRP reinforcing bars for concrete structures, Composite Structures, 78. 101-11.

Chen, Y. (2007), Accelerated ageing tests and long-term prediction models for durability of FRP bars in concrete, West Virginia University.

Eldridge, A. and Fam, A. (2014), Durability of concrete cylinders wrapped with GFRP made from furfuryl alcohol bioresin, Journal of Composites for Construction, 18, 04014013.

El-Hacha, R., Green, M.F. and Wight, G.R. (2010), Effect of severe environmental exposures on CFRP wrapped concrete columns, Journal of Composites for Construction, 14, 83-93.

Erdil, B., Akyüz, U. and Yaman, I.O. (2013), CFRP-confined concrete columns under different environmental conditions, Magazine of Concrete Research, 65, 731-43.

GangaRao, H.V., Taly, N. and Vijay, P. (2006), Reinforced concrete design with FRP composites, CRC press.

Guo, F., Al-Saadi, S., Singh Raman, R.K., Zhao, X. (2018), Durability of fiber reinforced polymer (FRP) in simulated seawater sea sand soncrete (SWSSC) environment, Corrosion Science, 141, 1-13.

Huiguang, Y., Yan, L., Henglin, L. and Quan, G. (2011), Durability of sea-sand containing concrete: Effects of chloride ion penetration, Mining Science and Technology, 21, 123-7.

Imran, I. and Pantazopoulou, S. (1996), Experimental study of plain concrete under triaxial stress, ACI Materials Journal, 93, 589-601.

Karbhari, V.M. (2007), Durability of Composites for Civil Structural Applications, Woodhead Publishing Limited, Cambridge, England.

Katsuki, F. and Uomoto, T. (1995), Prediction of Deterioration of FRP Rods Due to Alkali Attack. Non-Metallic (FRP) Reinforcement for Concrete Structures, Proceedings of the 2nd International RILEM Symposium, CRC Press.

Kshirsagar, S., Lopez-Anido, R.A. and Gupta, R.K. (2000), Environmental aging of fiber-reinforced polymer-wrapped concrete cylinders, ACI Materials Journal, 97, 703-12.

Li, Y.L., Zhao, X.L., Raman Singh, R.K. and Al-Saadi, S. (2016a), Experimental study on seawater and sea sand concrete filled GFRP and stainless steel tubular stub columns, Thin-Walled Structures, 106, 390-406.

Li, Y.L., Zhao, X.L., Raman Singh, R.K. and Al-Saadi, S. (2016b), Tests on seawater and sea sand concrete-filled CFRP, BFRP and stainless steel tubular stub columns, Thin-Walled Structures, 108, 163-84.

Li, Y.L., Zhao, X.L., Singh Raman, R.K. and Al-Saadi, S. (2018a), Thermal and mechanical properties of alkali-activated slag paste, mortar and concrete utilising seawater and sea sand, Construction and Building Materials, 159, 704-24.

Li, Y.L., Teng, J.G., Zhao, X.L. and Singh Raman, R.K. (2018b), Theoretical model for seawater and sea sand concrete-filled circular FRP tubular stub columns under axial compression, Engineering Structures, 160, 71-84.

Litherland, K., Oakley, D. and Proctor, B. (1981), The use of accelerated ageing procedures to predict the long term strength of GRC composites, Cement and Concrete Research, 11, 455-66.

Liu, H.-K., Tai, N.-H. and Lee, W.-H. (2002), Effect of seawater on compressive strength of concrete cylinders reinforced by non-adhesive wound hybrid polymer composites, Composites Science and Technology, 62, 2131-41.

Micelli, F., Mazzotta, R., Leone, M. and Aiello, M. (2014), Review study on the durability of FRP-confined concrete, Journal of Composites for Construction, 19, 04014056.

Micelli, F. and Myers, J. (2008), Durability of FRP-confined concrete, Proceedings of the Institution of Civil Engineers-Construction Materials, 161, 173-85.

Mohammed, T.U.T., Hamada, H. and Yamaji, T. (2004), Performance of seawater-mixed concrete in the tidal environment, Cement and Concrete Research, 34, 593-601.

Nishida, T., Otsuki, N., Ohara, H., Garba-Say, Z.M. and Nagata, T. (2015), Some considerations for applicability of seawater as mixing water in concrete, Journal of Materials in Civil Engineering, 7, B4014004-1-7.

Nkurunziza, G., Debaiky, A., Cousin, P. and Benmokrane, B. (2005), Durability of GFRP bars: A critical review of the literature, Progress in Structural Engineering and Materials, 7, 194-209.

Ozbakkaloglu, T., Lim, J.C. and Vincent, T. (2013), FRP-confined concrete in circular sections: Review and assessment of stress–strain models, Engineering Structures, 49, 1068-88.

Parnas, R., Shaw, M.T. and Liu, Q. (2007), Basalt fiber reinforced polymer composites, The New England Transportation Consortium.

Popovics, S. (1973), A numerical approach to the complete stress-strain curve of concrete, Cement and Concrete Research, 3, 583-99.

Provis, J.L. and Bernal, S.A. (2014), Geopolymers and related alkali-activated materials, Annual Review of Materials Research, 44, 299-327.

Robert, M. and Fam, A. (2012), Long-term performance of GFRP tubes filled with concrete and subjected to salt solution, Journal of Composites for Construction, 16, 217-24.

Saenz, N and Pantelides, C.P. (2006), Short and medium term durability evaluation of FRP-confined circular concrete, Journal of Composites for Construction, 10, 244-53.

Silva, M.A. (2007), Aging of GFRP laminates and confinement of concrete columns, Composite Structures, 79, 97-106.

Teng, J.G., Huang, Y.L., Lam, L. and Ye, L.P. (2007), Theoretical model for fiber-reinforced polymer-confined concrete, Journal of Composites for Construction, 11, 201-10.

Teng, J.G., Jiang, T., Lam, L. and Luo, Y.Z. (2009), Refinement of a design-oriented stress–strain model for FRP-confined concrete, Journal of Composites for Construction, 13, 269-78.

Teng, J.G., Yu, T., Dai, J.G. and Chen, G.M. (2011), FRP composites in new construction: current status and opportunities, in: Proceedings of 7th National Conference on FRP Composites in Infrastructure (Supplementary Issue of Industrial Construction), keynote presentation.

Toutanji, H. and Balaguru, P. (1998), Durability characteristics of concrete columns wrapped with FRP tow sheets, Journal of Materials in Civil Engineering, 10, 52-7.

Toutanji, H. and Saafi, M. (2001), Durability studies on concrete columns encased in PVC–FRP composite tubes, Composite Structures, 54, 27-35.

Toutanji, H.A. (1999), Durability characteristics of concrete columns confined with advanced composite materials, Composite Structures, 44, 155-61.

Walker, R.A. and Karbhari, V.M. (2007), Durability based design of FRP jackets for seismic retrofit, Composite Structures, 80, 553-68.

Wang, Z., Zhao, X.-L., Xian, G., Wu, G., Singh Raman, R.S. and Al-Saadi, S. (2018), Effect of sustained load and seawater and sea sand concrete environment on durability of basalt-and glass-fibre reinforced polymer (B/GFRP) bars, Corrosion Science.

Wang, Z., Zhao, X.-L., Xian, G., Wu, G., Singh Raman, R.S., Al-Saadi, S., et al. (2017), Long-term durability of basalt-and glass-fibre reinforced polymer (BFRP/GFRP) bars in seawater and sea sand concrete environment, Construction and Building Materials, 139, 467-89.

Xiao, J., Qiang, C., Nanni, A., Zhang, K. (2017), Use of sea-sand and seawater in concrete construction: Current status and future opportunities, Construction and Building Materials, 155, 1101-11.

Xie, Z., Xie, J., Guo, Y. and Huang, Y. (2018), Durability of CFRP-wrapped concrete exposed to hydrothermal environment, International Journal of Civil Engineering, 16, 527-41.

Yu, T., Teng, J. and Wong, Y. (2010), Stress-strain behavior of concrete in hybrid FRP-concrete-steel double-skin tubular columns, Journal of Structural Engineering, 136, 379-89.

Chapter 9

Conclusions and future work

9.1 Conclusions

This thesis had focused on the behaviour of hybrid tubular sections utilising seawater and sea sand concrete (SWSSC), glass/carbon/basalt fiber reinforced polymer (G/C/B-FRP) and stainless steel (SS). Both the short-term structural behaviour and durability performance were investigated by the means of experimental study and theoretical analysis. Based on the work conducted in this PhD thesis, this hybrid section could effectively utilise the pros of each individual materials and shows desirable short-term and long-term performances. The detailed conclusions are summarized as following.

Seawater and sea sand concrete

(1) Concrete mixture was proposed for alkali-activated slag-based seawater and sea sand concrete (SWSSC), which could reach the target strength (30~60 MPa) and desirable workability.

(2) SWSSC generally exhibits similar mechanical properties (i.e., strength, stress-strain curves) to ordinary Portland cement concrete utilising fresh water and river sand at room or elevated temperatures.

(3) The residual strengths of paste, mortar and concrete decrease with the increase of exposure temperature, and no strength gain was observed for slag paste in this research. The slag paste displays a rapid strength deterioration upon heating (from 100 °C), whilst the strength reduction of cement paste is less than 30% until the decomposition of portlandite (600 °C). On the other hand, the strength reduction trends of concretes are generally similar. The seawater, sea sand, and coarse aggregate with larger size slightly (less than 10%) reduce the residual strength. The samples become more deformable after heating and the residual Young's moduli drop more rapidly than residual strength when temperature is increased.

(4) the mechanical properties degradation of slag paste are mainly caused by cracks induced by temperature gradient and pore pressure and phase changes at high temperature, among which the cracks dominate the degradation. On the other hand, the main mechanism of the mechanical properties degradation of concrete, regardless using slag or cement, seawater or fresh water, river sand or sea sand, is the thermal expansion incompatibility between the contraction of paste matrix and expansion of aggregates. The influence of seawater and sea sand on the thermal properties is not obvious.

Seawater and sea sand concrete filled FRP and stainless steel tubes

(5) The strength and ductility of SWSSC-filled FRP and stainless steel tubes are significantly enhanced in comparison with hollow sectional tubes and plain concrete.

(6) Among the three FRPs (i.e., glass/carbon/basalt FRP) investigated, CFRP has the highest ultimate strength and elastic moduli in both longitudinal and transverse directions. BFRP has the lowest ultimate strength and elastic modulus in longitudinal direction, while the material properties of BFRP in hoop direction are similar to those of GFRP. The ultimate strain of CFRP is much less than that of BFRP and GFRP.

(7) The confinement effect provided by SS tubes is lower than that by FRP tubes, but the confinement can be maintained for larger axial strain for SS tubes. The strength enhancement and ductility of SWSSC-filled CFRP tubes are higher than those of GFRP and BFRP tubes mainly due to its higher hoop strength. The confinement provided by BFRP and GFRP tubes is quite similar.

(8) SWSSC-filled FRP tubes were failed by tube rupture in hoop direction and FRP tubes were progressively buckled in longitudinal direction before reaching this ultimate state. However, for some of the double-skin tubes with FRP as both the outer and inner tubes, the outer tube rupture did not occur as the inner FRP cannot effectively restrain the lateral expansion of concrete after the axial strain reaches around 0.03.

(9) SWSSC-filled FRP tube exhibits different dilation property to concrete-filled FRP wrap due to the biaxial stress state and Poisson effect in FRP tube. The proposed analysis-oriented models could reasonably predict the load-strain curves of SWSSC-filled FRP tubes (both fully filled and double-skin tubes) and the predictions are in good agreement with experimental data.

(10) Design formulas were proposed to estimate the ultimate capacity of SWSSC-filled stainless steel tubes and reasonable accuracy was achieved.

Durability

(11) Strength of seawater and sea sand concrete is not reduced after exposure to artificial sea water (3.5% NaCl solution). However, considerable hoop strength reduction occurs in GFRP and BFRP (23% and 39% loss respectively after 6 months in 40 °C artificial sea water) whereas the reduction for CFRP is only 8%. Environmental effect on Young's modulus of FRP is insignificant.

(12) Normalized strength (i.e., ratio of confined concrete strength-to-unconfined concrete strength, f_{cc}'/f_{c}') and ultimate strain of SWSSC-filled FRP stub columns decrease gradually with the increase of exposure duration. Durability performances of fully filled and double-skin tubes are similar and SWSSC-filled CFRP tubes perform superior than SWSSC-filled G/BFRP tubes in terms of residual strength.

(13) Strength degradation of stub columns is mainly caused by the deterioration of FRP material. Environmental aging leads to a partly effective confinement for columns with low confining stiffness. A confinement effectiveness factor is proposed in developing the load-axial strain curves to consider the environmental effect on confinement effectiveness.

(14) The testing data in this study confirmed that environmental reduction factor specified in design guideline (e.g. ACI 440.2R-08) for CFRP (i.e. 0.85) generally provides a safe estimation on the long-term strength of SWSSC-filled FRP tubes.

9.2 Future work

Recommendations are outlined as below which could be account for in future research on hybrid sections utilising seawater and sea sand concrete, FRP and stainless steel.

(1) It is suggested that high performance SWSSC be developed to achieve higher strength (i.e., >60 MPa) and better workability (e.g., self-consolidation concrete), which is more attractive in engineering practice.

(2) Only one type of laminate structure (i.e., [15/45/75]) for FRPs was investigated in the current study. As the fiber orientations could greatly affect the properties of FRP, it is suggested to study the short-term and long-term behaviour of FRP tubes with different laminate structures to clarify the effect of fiber orientations. It is also needed to investigate the influence of Poisson's ratio on the confinement effect.

(3) The proposed hybrid section could be adopted for beams (e.g., bending) and long columns (e.g., eccentric compression and global buckling), which are more practical in engineering applications. The structural behaviour of stub columns offers a base to investigate the more "complicated" members, such as beam-columns. Further research is recommended on SWSSC-filled FRP and stainless steel tubular beams, long columns and beam-columns

(4) Due to the limited time of PhD candidature, accelerated degradation tests were conducted to understand the durability performance of SWSSC-filled FRP tubes. However, there are some debates in using higher temperature or simulated solutions to accelerate the degradation process of FRP. Field demonstration test is suggested to investigate the durability performance of FRP in a real environment even though this kind of experiment may take a quite long time.

(5) Members in a real structure are subjected to service load but the effect has not been incorporated in the current study when assessing the durability performance. In the future, it is suggested to study the long-term behaviour of SWSSC-filled FRP and SS tubes under combined sustained loads and environmental aging. Furthermore, the experimental work in this thesis was based on small scale specimens. It is necessary to investigate the size effect of SWSSC under compression.