Shear behavior of deep precast/prestressed concrete hollow-core slabs with and without fibers at ambient and under fire conditions

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**SHEAR BEHAVIOR OF DEEP
PRECAST/PRESTRESSED CONCRETE HOLLOW-CORE
SLABS WITH AND WITHOUT FIBERS AT AMBIENT
AND UNDER FIRE CONDITIONS**

**NGUYEN THI NGUYET HANG
SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING**

2020

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AND UNDER FIRE CONDITIONS**

NGUYEN THI NGUYET HANG

School of Civil and Environmental Engineering

A thesis submitted to the Nanyang Technological University
in partial fulfilment of the requirement for the degree of
Doctor of Philosophy

2020

Statement of Originality

I hereby certify that the work embodied in this thesis is the result of original research, is free of plagiarised materials, and has not been submitted for a higher degree to any other University or Institution.

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Tan Kang Hai

Authorship Attribution Statement

This thesis contains material from 3 papers published in the following peer-reviewed journal / from papers accepted at conferences in which I am listed as an author.

Chapter 4 is published as Nguyen, T. N. H., K.-H. Tan and T. Kanda (2019). "Investigations on web-shear behavior of deep precast, prestressed concrete hollow core slabs." *Engineering Structures* **183**: 579-593

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The contributions of the co-authors are as follows:

- All experimental investigations and numerical simulations were conducted by me.
- I wrote the drafts of the manuscript.
- Prof Tan Kang Hai provided the initial research direction and revised the manuscript drafts.
- Dr. Kanda provided suggested some ideas to improve the experimental work and provided comments on the manuscript.

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- All experimental investigations and numerical simulations were conducted by me.

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The contributions of the co-authors are as follows:

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- I wrote the drafts of the manuscript.
- Prof Tan Kang Hai together with Dr. Kanda provided comments and revised the manuscript drafts.

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SUMMARY

Precast/prestressed concrete hollow core (PCHC) slabs have been the favorite products for wall or floor systems since the 1970s. Notwithstanding this, there are still some concerns regarding their web-shear capacity. The reason for this lies in the fact that the presence of longitudinal voids in PCHC slabs reduces the web thickness, resulting in a reduction in the shear capacity compared to that of solid slabs. Moreover, the extrusion method, which is the most popular method to fabricate PCHC slabs, does not allow PCHC slabs to incorporate any stirrups or shear reinforcement. Therefore, their web-shear capacity depends solely on the tensile strength of concrete, which is relatively low.

Previous studies have shown that PCHC slabs, deep PCHC slabs which have non-circular voids and flat webs with depths exceeding 300 mm could fail in web-shear mode at imposed loads lower than those calculated using ACI 318, Eurocode 2, and EN 1168. Some studies have been carried out to find out why deep PCHC slabs are more susceptible to web-shear failure. However, up to now, there has still been no satisfactory consensus. Under fire conditions, load-carrying capacity of PCHC slabs significantly decreases due to (1) a reduction in strength and stiffness of concrete and prestressing strands and (2) additional stresses induced by elevated temperatures. Previous studies have shown that when subjected to shear, fire-exposed PCHC slabs, especially deep ones, commonly failed at an early stage before achieving their design fire resistance. However, due to limited test data, the way deep PCHC slabs fail in web-shear under fire conditions has not been well understood.

To bridge the gap, this research was carried out, mainly aiming to (1) gain in-depth understanding of web-shear mechanisms of deep PCHC slabs at ambient and in the event of fire and (2) to investigate the potential use of polypropylene (PP) and steel fibers as a shear-strengthening technique to enhance web-shear capacity of PCHC slabs at both ambient and fire conditions.

To achieve the objectives, an experimental program consisting of 27 full-scale tests on 21 hollow-core specimens with and without fibers, divided into 4 series, was carried out at Nanyang Technological University, Singapore. While the first two series investigated shear behavior of conventional PCHC slabs at ambient (Series I) and under fire conditions (II), the last two series studied shear mechanisms of fiber-reinforced-concrete (FRC) hollow-core slabs at both ambient (Series III) and in the event of fire (Series IV). Besides the experimental studies, test data obtained from Series I, II, IV were used to verify finite element (FE) models developed to simulate shear behavior of PCHC slabs at ambient and under fire conditions. Numerical analyses based on the verified FE models were subsequently performed. The FE results, in turn, helped to shed light on shear behavior of PCHC slabs.

Based on results and observations from the experimental and numerical studies, two semi-analytical models were developed. The first model which is an extension of Yang (1994)'s model was developed to predict web-shear strength of conventional PCHC slabs. The proposed model adopted a rational failure criterion from Kupfer and Gerstle (1973) to account for the principal compression-tension stress state in the concrete webs (Chapter 5). When steel fibers are included, web-shear failure mode commonly changed to flexural-shear mode as shown in the experimental investigations. Thus, the second semi-analytical formula which is based on flexural-shear mechanism and takes into account the contribution of steel fibers was developed for steel-fiber-reinforced PCHC slabs (Chapter 7). The two models were then verified using test data from this study and from published literature. It is shown that the proposed models provide reasonable predictions for shear strength of PCHC slabs with and without steel fibers and can be adopted in design codes.

LIST OF PUBLICATIONS

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LIST OF SYMBOLS

A	concrete area;
A_{cp}	concrete cross-section area above the critical level;
A_p	area of prestressing steel;
A_f	cross-sectional area of steel fiber;
$C_{\theta,1}$	coefficient accounting for concrete stress under fire conditions;
$C_{\theta,2}$	coefficient accounting for anchored longitudinal reinforcement;
D_f	fiber diameter;
$F_{R,a,fi,p}$	force capacity of prestressing steel anchored in considered cross section;
F	$= (L_f/D_f)V_f d_f$ is fiber factor;
$F_{pullout}$	total pullout force of steel fibers;
I_y	moment of inertia;
L_t	transmission length;
L_f	fiber length;
$M_y(x_{cp})$	bending moment at the critical point;
N_p	effective prestress in strands;
N_{fib}	effective number of steel fibers across the crack plane;
R_y	reaction force at support;
S_y	first area moment of cross section;
$V_z(x_{cp})$	shear force at the critical point;
$V_{Rd,c}$	shear strength;

$V_{Rd,c,fi}$	fire shear strength in regions cracked in bending;
V_{cw}	web-shear capacity (ACI 318 code);
V_{ci}	flexural-shear capacity (ACI 318 code);
V_{SFRC}	shear strength of SFRC member;
V_f	volume fraction of steel fiber;
V_{fib}	shear resistance due to steel fibers;
b_w	minimum web thickness;
d	effective depth;
d_f	bond factor;
dN_p/dx	gradient of the prestressing force of strands;
k	size factor = $1 + (200/d)^{1/2}$ with d in mm, $k \leq 2$;
$k_p(\theta_p)$	strength reduction factor for the prestressing steel in Eurocode 2 Part 1-2;
f_c	compressive strength of concrete;
$f_{c,fi,m}$	average strength of concrete at elevated temperature;
$f_c(\theta)$	compressive strength of concrete at temperature θ ;
f_{ct}	tensile strength of concrete;
$f_{ct}(\theta)$	tensile strength of concrete at temperature θ ;
f_{py}	ambient yield strength of prestressing steel;
$f_{py}(\theta)$	yield strength of prestressing steel at temperature θ ;
f_{Ftuk}	characteristic value of the ultimate residual strength for FRC;
f_{spfc}	splitting-cylinder strength of SFRC;

x_{cp}	horizontal distance between the center of support and the critical point;
z_{cp}	vertical distance between the critical point and the neutral axis;
σ_x	horizontal normal stress;
σ_{cp}	full concrete compressive stress at the centroidal axis;
σ_p	the prestress of strand;
σ_{p0}	effective prestress of strand;
σ_1	maximum principal stress;
σ_2	minimum principal stress;
τ_{xz}	shear stress;
α_l	degree of prestressing transmission ($\alpha_l \leq 1$);
ρ	longitudinal reinforcement ratio;
ϕ	prestressing strand diameter;
γ_c	partial safety factor of concrete;
α	orientation factor account for the random orientation of fibers in an arbitrary cracking plane;

LIST OF ABBREVIATIONS

CDP model	Concrete damage plasticity model
FE	Finite element
FRC	Fiber-reinforced concrete
H fiber	Hooked steel fiber
LC	Load cell
LVDT	Linear variable displacement transducer
PCHC slabs	Precast/prestressed concrete hollow-core slabs
PP fiber	Polypropylene fiber
RC	Reinforced concrete
S fiber	High-strength/straight fiber
SEM	Scanning electron microscope
SFRC	Steel-fiber-reinforced concrete

CHAPTER 1 INTRODUCTION

1.1 General

Precast/prestressed concrete hollow-core (PCHC) slabs are widely used in residential, commercial and industrial buildings as floor or roof deck systems, or wall panels in concrete or composite structures (Fig. 1.1). They have been favorite products since the 1970s due to several significant advantages. Primarily, the presence of longitudinal voids in PCHC slabs allows weight and cost to reduce. As an added benefit, the voids can be utilized for concealed electrical or mechanical conduits in buildings. Moreover, with an efficient prestressing design, lower self-weight, longer span lengths (up to 18 m in simply supported systems), and higher sustained loads can be achieved. Further, speedier construction, assured quality, and completely self-supporting are other benefits that PCHC slabs bring about.

Fig. 1.1 Hollow-core floor systems; Courtesy of picture a): <http://tropicalconcrete.com>;
Courtesy of pictures b) and c): <http://www.jpcarrara.com>

PCHC slabs can be produced by a number of methods, e.g. slip-forming, wet casting, or extrusion on long line prestressing beds, so-called the extrusion method (Yang 1994). The two most common techniques are the slip-forming and the extrusion method while the latter is becoming more and more widespread. The manufacturing process of the two methods comprises three basic steps. First, prestressing strands are tensioned above a long bench (100 to 200 m in length). Subsequently, concrete is cast automatically by a molding and casting machine that moves along the bench. Finally, after hardening of the concrete, the strands are released, and the long slabs are sawn cut into desired lengths (Fig. 1.2). While a very low (zero or nearly zero) slump concrete is extruded through the casting machine in the extrusion method, a higher slump concrete is used in the slip-forming system. However, both methods restrict the placement of shear reinforcement.

Fig. 1.2 Manufacturing process of PCHC slabs (the extrusion method)

Typically, precast hollow-core slabs are fabricated with a standard width of 1.2 m while the length and thickness vary. Current slab thickness ranges from 100 to 500 mm with different core configurations. Recently, with the development of technology and an

increase in demand from the construction industry, deep PCHC slabs characterized by non-circular voids with depths of more than 300 mm have gained great interests. Fig. 1.3 depicts cross sections of a shallow PCHC slab with circular voids and a deep PCHC slab with non-circular voids.

Fig. 1.3 Cross sections of shallow and deep PCHC slabs

1.2 Problem statement

1.2.1 PCHC slabs at ambient condition

With a reduction in cross section due to the presence of the longitudinal voids, relatively low tensile strength of concrete, and most importantly, the absence of stirrups, PCHC slabs are naturally susceptible to web-shear failure. This susceptibility is even more pronounced for deep PCHC slabs, distinguished by non-circular voids and flat webs with depths greater than 300 mm (Fig. 1.3(b)). Previous studies have shown that deep PCHC slabs could fail in web-shear at loads less than those predicted by common design codes such as ACI 318 and Eurocode 2 (Walraven and Mercx 1983, Hawkins and Ghosh 2006, Palmer and Schultz 2011, Brunesi, Bolognini et al. 2015, Nguyen, Tan et al. 2019).

Recognizing deep PCHC slabs are more susceptible to web-shear failure than shallow units, ACI 318-08 (Committee, Institute et al. 2008), as well as the later version, i.e. ACI 318-14 (ACI 2014), has implemented the minimum shear-reinforcement requirement for PCHC slabs with depths greater than 12.5 in (318 mm), if the shear load (factored) at Ultimate Limit State is larger than 50% of the design shear strength of concrete. When placing shear reinforcement is not possible, the web-shear capacity must be reduced by 50%. However, up to now, there has been no equivalent restriction imposed on Eurocode

2 (EN-1992-1-1 2004). Nonetheless, it is noteworthy that the design of PCHC slabs in Europe nowadays follows the standard EN 1168:2005+A3:2011 (EN-1168:2005+A3 2011), i.e. an extension of Eurocode 2 specifying for hollow-core slab products. Unlike Eurocode 2, EN 1168:2005+A3:2011 design code has introduced some reduction factors for the web-shear capacity of PCHC slabs.

Along with the changes in the design codes, some studies have been carried out to find out why deep PCHC slabs are more susceptible to web-shear failure (Yang 1994, Palmer and Schultz 2011, Brunesi, Bolognini et al. 2015). However, suffice to say, there has still been no satisfactory or wide-held consensus. As deep PCHC slabs have gained greater usage in industrial buildings, more studies on web-shear failure mechanisms of such slabs are urgently needed.

1.2.2 PCHC slabs under fire conditions

Fire resistance of PCHC slabs is currently determined through standard fire tests or the prescriptive method specified in Eurocode 2 (EN-1992-1-2 2004), of which fire resistance is ensured by providing minimum values of slab thickness and axis distance (the distance from prestressing strand to the nearest exposed soffit). However, data from Jansze, Acker et al. (2012) revealed that although PCHC slabs were designed to sustain at least 1h fire rating, in more than half of the cases, the slabs failed prematurely in web-shear mode before achieving their design fire resistance (Fig. 1.4), showing that the prescriptive method is not conservative for PCHC slabs subjected to shear. It is confirmed that shear is a great concern when using this type of slabs, in particular, deep PCHC slabs with an overall depth greater than 300 mm.

Fig. 1.4 Fire resistance time of 42 fire tests (from Jansze, Acker et al. (2012))

Fire behavior of PCHC slabs subjected to fire is governed by temperature-dependent material properties of concrete and prestressing strands, and stresses induced by temperature (Borgogno and Fontana 2000, Fellingner 2004). It has been shown that under fire conditions, the web-shear capacity of hollow-core slabs significantly decreases due to (a) degradation in strength and stiffness of the two materials and (b) additional tensile stress in the concrete webs caused by a thermal gradient over slab depth, which is more pronounced for hollow-core slabs with depths exceeding 300 mm. However, most fire shear tests in the literature were conducted on shallow PCHC slabs with circular voids and depths smaller than 300 mm (Andersen and Lauridsen 1999, Borgogno and Fontana 2000, Fellingner 2004). To date, there are only four fire shear tests performed on deep PCHC slabs (with a depth of 400 mm, shown in Fig. 1.4), in which three were reduced-scale tests, carried out on double-web specimens sawn from a standard hollow-core unit of 1.2 m wide. As a result, these fire tests did not provide sufficient information for an in-depth study on shear response of PCHC slabs subjected to fire. Consequently, the way fire-exposed PCHC slabs fail in web-shear has not been well established. Thus, there is an urgent need to shed light on the shear behavior of PCHC slabs under fire conditions to ascertain adequate safety when using this product.

1.2.3 Fiber-reinforced concrete (FRC) hollow-core slabs

As stirrups cannot be placed in PCHC slabs and shear failure is critical, several techniques to strengthen shear resistance of hollow-core slabs have been investigated, e.g. applying core filling (Palmer and Schultz 2011), using fiber-reinforced polymer composite sheets (Wu, El Ragaby et al. 2016), or using polypropylene (PP) or steel fibers (Peaston and Paine 1999, Palmer and Schultz 2011, Cuenca and Serna 2013, Simasathien and Chao 2015, Dudnik, Milliman et al. 2017). Among them, the last method has attracted the most attention. However, the number of studies on using steel fibers in hollow-core slabs is still rather limited. In addition, the amount of fiber needed to achieve the desired effects has not been clarified. Furthermore, the use of fibers to enhance shear resistance of PCHC slabs has only been studied at room temperature but no study has been carried out under fire conditions. As a result, fire behavior of FRC hollow-core slabs has not been investigated yet and the effectiveness of using fibers on PCHC slabs under fire conditions could not be quantified.

1.3 Research objectives, scope of work and research significance

Objectives of research

This research is carried out to address the above-mentioned technical gaps in research and limitations of code provisions, aiming to:

- 1) Gain an overall understanding of web-shear behavior of deep PCHC slabs at ambient and under fire conditions.
- 2) Develop robust numerical models simulating web-shear responses of the slabs at ambient and under elevated temperatures. These models, in turn, could be used to extend understanding of failure mechanisms of PCHC slabs and clarify why deep hollow-core slabs are more susceptible to web-shear failure compared to others.
- 3) Investigate structural responses of FRC hollow-core slabs at ambient and in the event of fire. The effectiveness of using fibers to enhance shear resistance of PCHC slabs can then be quantified.

- 4) Use the test data to re-evaluate the validity of code provisions applied to PCHC slabs including ACI 318-14, Eurocode 2, EN 1168:2005+A3:2011.
- 5) Develop semi-empirical approaches to predict shear strength of hollow-core slabs with and without steel fibers.

Scope of work

To achieve the five objectives, an experimental program consisting of 21 specimens, divided into 4 series, were conducted. The testing program is summarized in Fig. 1.5. Numerical and analytical studies were subsequently carried out. Tasks implemented in this research program are as follows:

- In Series I, 4 full-scale shear tests on PCHC slabs with depths ranging from 320 mm to 500 mm were conducted. Shear-strength results of the 4 specimens were compared to those calculated by design codes. Results from the experiments were also used to verify an FE model aimed to simulate web-shear behavior of PCHC slabs. Parametric studies using the verified FE model were then performed.
- Further, another five shear tests on hollow-core specimens similar to those tested in Series I were conducted under fire conditions (Series II). Effect of slab thickness and load level on web-shear behavior of deep PCHC slabs was investigated. Data obtained from the fire tests provided valuable information for an in-depth study on web-shear failure mechanisms of deep PCHC units exposed to fire. In addition, they were used to verify two FE models developed to simulate thermal and structural responses of PCHC slabs subjected to fire. The FE models, in turn, were used to extend the existing database of deep PCHC slabs subjected to fire.
- Thereafter, in Series III, twelve shear tests conducted on six fiber-reinforced hollow-core specimens were carried out. Effect of PP and steel fibers with different volume fractions on shear resistance of PCHC slabs was investigated. Test results from this series served as a basis for six other tests in the last series (Series IV), which employed similar specimens but were tested under fire conditions. Experimental and numerical results from Series IV helped to shed light on behavior of FRC hollow-

core slabs under fire conditions and to evaluate the effectiveness of different types of fiber on shear resistance of fire-exposed FRC slabs.

- Semi-empirical models based on the acquired experimental and numerical results are developed. The goal is to develop simplified approaches to predict ultimate web-shear strength of hollow-core slabs without any fibers (conventional slabs) and with steel fibers.

Fig. 1.5 Testing program

Contributions and novelties

Contributions and novelties of the research are as follows:

- 1) The study provides valuable experimental data and numerical investigations on shear behavior of deep PCHC slabs at ambient and under elevated temperatures, which helps to gain a more comprehensive understanding of web-shear behavior of the slabs.
- 2) The research contributes experimental test results and an overall assessment on the use of fibers including PP and steel fibers to strengthen shear capacity of extruded, deep PCHC slabs, not only at ambient but also under fire conditions.
- 3) A novel semi-analytical approach to assess web-shear strength of PCHC slabs. In the model, a more rational failure criterion compared to that used in traditional design approaches is adopted.
- 4) A semi-empirical formula to predict shear strength of steel-fiber-reinforced concrete (SFRC) hollow-core slabs is proposed.

1.4 Layout of the thesis

A total of nine chapters is included in this thesis; its layout is shown in Fig. 1.6. The first chapter provides an overall background of this study as well as motivations to conduct tests on web-shear behavior of deep PCHC slabs. Research objectives, scope of work, and deliverables are presented in Chapter 1.

In Chapter 2, an extensive literature review of published studies relevant to structural behavior of PCHC slabs is presented to show knowledge gaps in research. The first part focuses on failure modes, shear theory, and a review of published studies on shear behavior of deep PCHC slabs at room temperature. Thereafter, temperature-dependent material properties of concrete and prestressing steel and a comprehensive review of experimental, numerical and analytical studies on shear resistance of hollow-core slabs under fire conditions are discussed. Subsequently, techniques to enhance shear resistance of the units are reviewed. Finally, technical gaps in existing research are identified.

Chapter 3 presents an experimental program on deep PCHC slabs with and without fibers to address the major gaps found in the literature review in Chapter 2. An overview of the testing program including geometric and material properties of specimens, parameters, test setup, instrumentation, as well as heating conditions, is provided.

In Chapter 4, experimental and numerical investigations on web-shear behavior of deep PCHC slabs at ambient temperature (Series I) are presented. Based on observations from Chapter 4, a semi-analytical approach to design conventional PCHC slabs against web-shear failure is proposed and presented in Chapter 5. In the approach, a biaxial principal compression-tension failure criterion instead of maximum principal stress theory is adopted.

Thereafter, Chapter 6 presents experimental studies on shear response of deep PCHC slabs under fire conditions (Series II). Results from one heat transfer and four structural shear tests are discussed in this chapter. After the experimental studies, numerical investigations using Abaqus/Explicit (2014) and Concrete Damage Plasticity model (CDP model) are subsequently discussed, followed by code comparison.

As web-shear is critical for PCHC slabs, Chapter 7 investigates the use of PP and steel fibers to enhance shear capacity of hollow-core slabs (Series III). Twelve full-scale tests on six specimens with and without fibers are conducted at room temperature. A semi-empirical model to predict shear capacity of SFRC hollow-core slabs is proposed and presented in this chapter.

Chapter 8 presents experimental and numerical studies on the use of PP and steel fibers to enhance shear resistance of PCHC slabs under fire conditions. Temperature and deflection developments, crack patterns and failure modes of six specimens in Series IV are well discussed. Images from scanning electron microscope (SEM) are included to illustrate changes in microstructures of concrete before and after heating to study temperature-induced spalling effect.

Finally, Chapter 9 summarizes conclusions from all accomplished works. Recommendations for practice and future work are also presented.

Fig. 1.6 Thesis layout

CHAPTER 2 LITERATURE REVIEW

2.1 Introduction

A comprehensive review of published studies on shear behavior of precast/prestressed concrete hollow core (PCHC) slabs at ambient and under fire conditions is presented in this chapter. The main purpose is to identify and define objectives and to form the scope of work for this research. The first section (Section 2.2) focuses on shear studies on PCHC slabs at room temperature. In this section, possible failure modes, shear theories and previous investigations on shear behavior of PCHC slabs are reviewed and discussed. The second section (Section 2.3) reviews structural behavior of PCHC slabs under fire conditions. Temperature-dependent material properties of concrete and prestressing steel are first introduced. These properties are of great importance for assessment of structural fire analysis. Thereafter, failure modes of PCHC slabs under elevated temperatures are discussed. Subsequently, an extensive literature review of experimental, numerical, and analytical studies on fire resistance of PCHC slabs is presented. The last section (Section 2.4) discusses techniques to enhance shear capacity of PCHC slabs with an emphasis on the use of fibers. The chapter ends with salient points of knowledge gaps in existing research.

2.2 PCHC slabs at room temperature

2.2.1 Failure modes

To study failure mechanisms of PCHC slabs when overloaded, Walraven and Mercx (1983) conducted 25 tests at room temperature and observed four principal failure modes, i.e. flexural, anchorage, shear-compression, and shear-tension failures. They are reviewed in the following sections (2.2.1.1 to 2.2.1.4).

2.2.1.1 Flexural failure

Flexural failure is the most common failure mode in concrete structures. This mode is preferable because it is ductile and much more predictable compared to shear failure. In PCHC slabs, due to relatively low reinforcement ratio (commonly within the range of 0.5 to 1.5%), high ductility with large crack openings and deformation after first flexural cracks can be expected. Flexural failure then occurs with yielding of prestressing steels, followed by rupture of strands prior to concrete crushing (Fig. 2.1).

Fig. 2.1 Flexural failure in PCHC slabs (Walraven and Mercx 1983)

2.2.1.2 Anchorage failure

In a prestressed concrete member, prestressing force must be transferred from the strand/wire/tendon to concrete, and it is achieved by the bond at the interface between the prestressing steel and the concrete at the end zone. After the transfer, the strand presumably has zero stress at its two free ends and achieves an effective stress f_{pe} after a strand length from the two slab edges. The distance required for the effective stress to develop is called transmission length l_t (Fig. 2.2). However, to mobilize ultimate tensile strength for the strand f_{ps} ($f_{ps} > f_{pe}$), an additional bond length called the flexural bond length l_f is required. The flexural bond length l_f is added to the transmission length l_t resulting in the-so-called development length l_d (Naaman 2004). Hence, the development length can be interpreted as the distance that the strand requires to develop its ultimate strength (full design strength) f_{ps} . Generally, the transmission length and development length are about 50 and 150 times the nominal diameter of the strand, respectively. Fig.

2.2 illustrates the transmission, flexural, and development lengths in prestressed concrete members.

Fig. 2.2 Assumed variation of strand stress with distance from its free end

For a particular loading condition, if a large bending moment appears in the development length of a PCHC slab, anchorage failure may occur. As flexural cracks grow within the end zone, bond stress (or frictional stress) needed to anchor the strands reduces. If the degradation of bond stress is severe, the slab collapses as the strands slip through the concrete, inducing considerable crack widening and anchorage failure (Fig. 2.3).

Fig. 2.3 Anchorage failure (Walraven and Mercx 1983)

If flexural cracks occur within the transmission length, the strands are immediately pulled out from the concrete due to the high requirement for bond stress at the transmission length. Nevertheless, if the flexural cracks occur within the flexural bond length l_f , the strands are not pulled out immediately but after further increments of bending moment.

2.2.1.3 Shear-compression failure

Shear failure may arise from a flexural crack at bending regions, the so-called shear-compression failure or flexural-shear failure. In this mode of failure, flexural cracks are first formed when tensile stress in concrete at the bottom slab exceeds tensile strength of concrete. Due to a further increment of imposed load and a combined effect of flexure and shear, the cracks propagate diagonally towards the applied load location. An increase of the applied load subsequently results in failure of the compression zone, usually by crushing. If a PCHC slab is subjected to a concentrated load, shear-compression failure will occur in the vicinity of the applied load. However, if the applied load is uniformly distributed, failure occurs at the section where shear-compression capacity is exceeded (Walraven and Mercx 1983) (Fig. 2.4).

Fig. 2.4 Flexural-shear or shear-compression failure

2.2.1.4 Web-shear failure

Shear failure can occur from a crack forming in the web of a PCHC slab, near the bearing zone or where flexural cracks hardly develop. In this region, if shear stress due to the applied load causes the principal tensile stress in the web reaching tensile strength of concrete, an inclined crack occurs. Immediately, the crack propagates diagonally towards the imposed load and bearing regions, causing a sudden/brittle failure with formation of a web-shear crack, the so-called web-shear failure. Fig. 2.5 illustrates the web-shear failure in PCHC slabs.

Fig. 2.5 Shear-tension failure (or web-shear failure) (Becker and Buettner 1985)

2.2.2 Shear strength of PCHC slabs

2.2.2.1 Shear theory

Shear resistance of concrete members without stirrups such as PCHC slabs is primarily provided through three components: concrete in the compression region, aggregate interlock and dowel action of longitudinal reinforcement. In concrete slabs with very low reinforcement ratios such as hollow-core slabs, the contribution of dowel action is trivial compared to the contribution of the other two. Between the compression action and the aggregate interlock, some authors emphasized on the contribution of compression action over aggregate interlock (Bresler and Pister 1958, Tureyen and Frosch 2003) while some others believed that shear resistance primarily comes from the aggregate interlock action (Mitchell and Collins 1974, Vecchio and Collins 1986). For example, Bresler and Pister (1958) developed a shear model using the concept of average normal and shear stresses in the concrete compression region to determine shear capacity of RC beams. The model and calculation procedure are illustrated in Fig. 2.6.

$\bar{\tau} = \frac{V_u}{A_e} \quad \bar{\sigma} = \frac{C}{A_e}$ $\bar{\tau} = 0.1f_c \left[0.62 + 7.86 \left(\frac{\bar{\sigma}}{f_c} \right) - 8.46 \left(\frac{\bar{\sigma}}{f_c} \right)^2 \right]^{\frac{1}{2}}$ $V_u = \bar{\tau} A_e = \bar{\tau} b(kd)$	<p>$\bar{\tau}$ and $\bar{\sigma}$: Average shear and normal stress in the compression region, respectively;</p> <p>A_e: Effective area in the compression region resisting shear;</p> <p>C: Compressive force in the compression region;</p> <p>V_u: Ultimate shear strength</p>
---	---

Fig. 2.6 Shear model for RC beams from Bresler and Pister (1958)

On the other hand, Vecchio and Collins (1986) published a study named the Modified Compression Field Theory for reinforced concrete subjected to shear using the concept of average stress and strain in cracked concrete. In the model, shear stress at a crack is a function of aggregate size, crack width, and the compressive stress transferred through the crack. The Modified Compression Field Theory has been adopted in several design codes such as AASHTO LRFD bridge construction specifications and Canadian Concrete Code. Fig. 2.7 presents stress-strain relationship for cracked concrete and the proposed shear formula from the Modified Compression Field Theory.

Fig. 2.7 Stress-strain relationship for cracked concrete and shear calculation from Vecchio and Collins (1986)

In PCHC slabs, shear strength is commonly assessed through web-shear and flexural-shear resistance. While the web-shear capacity is calculated using theories from Strength of Materials, the flexural-shear capacity is predicted empirically. Besides the contribution of concrete and dowel action, compression due to prestressing force is also accounted for. The next section reviews some calculations for shear strength of PCHC slabs.

2.2.2.2 Shear strength of PCHC slabs

Web-shear capacity

As mentioned in Section 2.2.1.4, the web-shear failure occurs when the principal tensile stress in the concrete webs exceeds tensile strength of concrete. A diagonal crack occurs and immediately propagates in both directions toward the bearing region and the applied-load location, causing a sudden failure with the formation of a web-shear crack. In traditional design approaches, the critical point where a web-shear crack first initiates is supposed to be at the mid height of the web, which is close to neutral axis level (Fig. 2.8(a)). In the longitudinal direction, the critical point is postulated to be at a location of $h/2$ from the centerline of the support towards the slab center (Fig. 2.8(b)). Here, h denotes the thickness of the slab. With this assumption, the critical point is located outside the region where the vertical support pressure is active, but still within the development length where the prestressing force is not yet fully developed (Walraven and Mercx 1983).

Fig. 2.8 Critical point in horizontal and vertical directions as shown in Yang (1994)

While Fig. 2.9 shows the horizontal normal stress σ_x in a PCHC slab caused by the prestressing force (N_p) and the bending moment (M_y) due to an applied load (P), Fig. 2.10 shows the shear stress τ_{xz} caused by the shear force (V) due to the applied load. The biaxial principal compression-tension stress state at the critical point of the slab is presented in Fig. 2.11.

 Fig. 2.9 Horizontal normal stress σ_x due to prestressing force and bending moment

 Fig. 2.10 Shear stress τ_{xz} due to shear force

Fig. 2.11 Stress state at the critical point

From the elementary beam theory presented in Hibbeler's book (Hibbeler 2014), the horizontal normal stress σ_x and the shear stress τ_{xz} at the critical point can be derived:

$$\sigma_x = -\frac{N_p(x_{cp})}{A} + \frac{N_p(x_{cp})e}{I_y} z_{cp} - \frac{M_y(x_{cp})}{I_y} z_{cp} \approx -\frac{N_p(x_{cp})}{A} \quad (2.1)$$

$$\tau_{xz} = \frac{S_y(z_{cp})}{b_w(z_{cp})I_y} V_z(x_{cp}) \quad (2.2)$$

where

x_{cp} is horizontal distance between the center of support and the critical point;

$N_p(x_{cp})$ is the effective prestress in strands at the critical point,

A is concrete area;

- e is vertical distance between the centroid of strands and the neutral axis;
 I_y is moment of inertia;
 z_{cp} is vertical distance between the critical point and the neutral axis;
 $M_y(x_{cp})$ is bending moment at the critical point;
 $V_z(x_{cp})$ is shear force at the critical point;
 $S_y(z_{cp})$ is first area moment of cross section with respect to the neutral axis;
 $b_w(z_{cp})$ is web thickness at the critical point.

As can be seen in Fig. 2.8, the distance between the neutral axis and the critical point in the vertical direction z_{cp} is relatively small. Therefore, the terms $\frac{N_p(x_{cp})e}{I_y}z_{cp} - \frac{M_y(x_{cp})}{I_y}z_{cp}$ could be ignored in Eq. 2.1. Web-shear failure occurs when the principal tensile stress at the critical point σ_1 (Fig. 2.11) reaches the tensile strength of concrete f_{ct} , resulting in Eq. 2.3:

$$f_{ct} = \sigma_1 = \frac{\sigma_x}{2} + \frac{1}{2}\sqrt{\sigma_x^2 + 4\tau_{xz}^2} \quad (2.3)$$

Substituting Eq. 2.1 and 2.2 into Eq. 2.3:

$$f_{ct} = -\frac{N_p(x_{cp})}{2A} + \frac{1}{2}\sqrt{\frac{N_p^2(x_{cp})}{A^2} + 4\frac{S_y^2(z_{cp})}{b_w^2(z_{cp})I_y^2}V_z^2(x_{cp})} \quad (2.4)$$

The web-shear capacity is then calculated as follows:

$$V_z(x_{cp}) = \frac{b_w(z_{cp})I_y}{S_y(z_{cp})}\sqrt{f_{ct}^2 + \frac{N_p(x_{cp})}{A}f_{ct}}$$

$$V_z(x_{cp}) = \frac{b_w(z_{cp})I_y}{S_y(z_{cp})}\sqrt{f_{ct}^2 + \sigma_{cp}f_{ct}} \quad (2.5)$$

Eq. 2.5 is the original form of formulas to calculate web-shear capacity of a single span beam/slab in regions uncracked by bending in commonly known design codes such as ACI 318-14 (ACI 2014) and Eurocode 2 (EN-1992-1-1 2004). Particularly, web-shear capacity following EN 1992-1-1: 2004 is given as:

$$\text{EN 1992-1-1} \quad V_{Rd,c} = \frac{l \cdot b_w}{S} \sqrt{(f_{ctd})^2 + \alpha_l \sigma_{cp} f_{ctd}} \quad (\text{Eq. 6.4})$$

EN 1168:2005+A3:2011, an extension of Eurocode 2 for the design of PCHC slabs, introduces two reduction factors, i.e., φ and β for the web-shear resistance of the slabs. While β is the reduction factor for transmission length, φ accounts for secondary shear stresses owing to transmissions of prestressing force adopted from Yang (1994). The web-shear strength of PCHC slabs following EN 1168:2005+A3:2011 is calculated as:

$$\text{EN 1168:2005+A3:2011} \quad V_{Rd,c} = \varphi \frac{I \cdot b_w}{S} \sqrt{(f_{ctd})^2 + \beta \alpha_l \sigma_{cp} f_{ctd}} \quad (\text{Sec. 4.3.3.2.2.3})$$

where

$\varphi = 0.8$ is reducing factor;

I is the second moment area (mm^4);

b_w is the minimum web thickness (mm);

S is the first moment of area above and about the centroidal axis (mm^3);

f_{ctd} is the design value of tensile strength of concrete (MPa);

$\beta = 0.9$ is reducing factor referred to transmission length;

α_l is the degree of prestressing transmission ($\alpha_l \leq 1$);

σ_{cp} is the full concrete compressive stress at the centroidal axis (MPa).

Finally, the ACI 318-14 web-shear strength is given by Eq. 22.5.8.3.2 as follows. It should be noted that for PCHC units with depths more than 12.5 in (318 mm), the minimum shear-reinforcement requirement is applied if the factored shear force at Ultimate Limit State is greater than 50% of the design shear capacity of concrete. Where the placement of shear reinforcement is not fulfilled, the web-shear capacity calculated by Eq. 22.5.8.3.2 must be reduced by 50%.

$$\text{ACI 318-14} \quad V_{cw} = (3.5\lambda\sqrt{f_c} + 0.3f_{pc})b_wd + V_p \quad (\text{Eq. 22.5.8.3.2})$$

where

V_{cw} is the web-shear capacity (lb.);

f_c is specified compressive strength of concrete (psi);

f_{pc} is compressive stress in concrete due to the effective prestress (psi);

b_w is the net web width of hollow-core slab (in.);

d is the effective depth of hollow core slab (in.);

V_p is vertical component of effective prestress force at section (lb.).

Flexural-shear capacity

Flexural-shear capacity formulas are derived empirically, considering the contribution of concrete, dowel action through longitudinal reinforcement, and compression force due to prestressing practice. Calculations of EN 1168:2005+A3:2011 for the flexural-shear strength at cracked regions by bending follow EN 1992-1-1: 2004 (Sec. 6.2.2) as:

$$\text{EN 1992-1-1} \quad V_{Rd,c} = [C_{Rd,c} k (100 \rho_1 f_{ck})^{\frac{1}{3}} + k_1 \sigma_{cp}] b_w d \quad (\text{Eq. 6.2.a})$$

$$\text{with a minimum of} \quad V_{Rd,c} = (v_{\min} + k_1 \sigma_{cp}) b_w d \quad (\text{Eq. 6.2.b})$$

where

$V_{Rd,c}$ is the design value for the shear resistance (N);

$C_{Rd,c}$ and k_1 are recommended to be 0.12 and 0.15, respectively;

$k = 1 + \sqrt{\frac{200}{d}} \leq 2$ with d is effective depth of a cross-section (mm);

ρ_1 is the longitudinal reinforcement ratio;

f_{ck} is characteristic compressive cylinder strength of concrete (MPa);

σ_{cp} is axial stress in section due to prestress (MPa);

b_w is the smallest web width (mm);

$$v_{\min} = 0.035 k^{3/2} f_{ck}^{1/2}$$

From ACI 318-14, the flexural-shear capacity is the greater of:

$$\text{ACI 318-14} \quad V_{ci} = 0.6\lambda\sqrt{f_c} b_w d + V_d + \frac{V_i M_{cre}}{M_{\max}} \quad (\text{Eq. 22.5.8.3.1a})$$

$$\text{ACI 318-14} \quad V_{ci} = 1.7\lambda\sqrt{f_c} b_w d \quad (\text{Eq. 22.5.8.3.1b})$$

where

V_{ci} is the flexural-shear capacity (lb.);

V_d is shear force at section due to unfactored dead load (lb.);

$V_i = V_u - V_d$ is shear force at section due to externally applied loads (lb.);

M_{cre} is moment causing flexural cracking due to externally applied loads (in.-lb.);

$M_{\max}$ is maximum moment due to externally applied loads (in.-lb.);

λ is modification factor;

In general, shear capacity at each section of a PCHC slab is assessed through both web-shear and flexural-shear capacities. The lower value is taken as the shear capacity of that section.

2.2.3 Previous studies on shear behavior of deep PCHC slabs

Hawkins and Ghosh (2006) collected data from 28 deep PCHC slabs (ranging from 320 mm to 500 mm in thickness) manufactured using the extrusion method and tested by three different suppliers in the US. The study showed that out of 28 tests, 12 specimens failed in web-shear mode at an average of 67% (ranging from 53% to 83%) of the loads predicted by ACI 318-05 (ACI 2005). The average value for another 8 slabs was 85% (ranging from 74% to 102%). Similarly, Brunesi, Bolognini et al. (2015) compared shear strength results from 49 PCHC slabs tested in the past with shear capacity predicted by ACI 318 and Eurocode 2. The comparisons showed that ACI 318-05 overpredicted shear capacity for most of the specimens, whereas Eurocode 2 (EN-1992 2004) overestimated the web-shear capacity of PCHC slabs with a depth of 400 mm and 500 mm.

Recognizing deep PCHC slabs are more susceptible to web-shear failure compared to shallow slabs, a number of studies have been carried out to reveal underlying reasons.

For instance, Yang (1994) developed finite element (FE) models using ABAQUS (1989) with brick elements to model the behavior of concrete. The author observed that for PCHC slabs with flat webs and non-circular voids, the maximum principal tensile stresses were not located at the centroid of slabs as assumed by traditional design codes such as ACI 318 and Eurocode 2, but at the junction of the lower flange and the slenderest web thickness. Moreover, the induced shear stress in concrete webs caused by pretensioned strands was not taken into account in ACI 318 or Eurocode 2. The author proposed a new approach to bridge the shortcoming. Nonetheless, Brunesi, Bolognini et al. (2015) showed that for many cases, Yang's approach was still unconservative.

Palmer and Schultz (2011) carried out an experimental program consisting of 24 tests on deep PCHC slabs. The study showed that the phenomenon in which deep PCHC slabs failed at imposed loads less than those calculated using design codes was not necessarily related to size effect. In fact, poor compaction of concrete with zero slump caused by the extrusion method resulted in large end slip of strands even before loading was applied. Such a factor may be responsible for the over-prediction of ACI 318-05, i.e., the code which Palmer and Schultz (2011) used when conducting the research.

The review of the literature showed that although there are some studies conducted to find out why deep PCHC slabs are more susceptible to web-shear failure, until now, there is still no wide-held consensus. Therefore, more studies on web-shear behavior of deep PCHC slabs are needed to make the design safe.

2.3 PCHC slabs under fire conditions

2.3.1 Material properties at elevated temperatures

PCHC slabs consist of concrete and prestressing steel. Under elevated temperatures, thermal, mechanical, and deformation properties of the two materials degrade continuously. The thermal properties comprise thermal conductivity, specific heat, and density. They govern heat-transfer behavior of the slabs. On the other hand, mechanical

and deformation properties include stress-strain relationships, elastic modulus, and thermal elongation, which control the degradation in strength and stiffness, as well as deflection of PCHC slabs in the event of fire. The temperature-dependent properties of concrete and strands must be fully understood for an accurate structural fire analysis. Sections 2.3.1.1 and 2.3.1.2 respectively review thermal and mechanical properties of concrete and prestressing steels exposed to elevated temperatures. The review only focuses on changes in properties of normal-strength concrete with $f_c \leq 50$ MPa as high-strength concrete is outside the scope of this research.

2.3.1.1 Concrete properties under elevated temperatures

2.3.1.1.1 Thermal properties

Thermal properties of concrete contain thermal conductivity, specific heat, and density. Thermal conductivity is the property of a material to conduct heat. It is described as the quantity of heat transmitted through a unit thickness in the direction perpendicular to the exposed surface due to a unit rise in temperature. Specific heat, on the other hand, is the amount of heat per unit mass required to raise one-unit temperature. Thermal properties of concrete depend on aggregate type and moisture content. They are not constant but vary with an increase of temperature. Fig. 2.12 to Fig. 2.14 show thermal properties of concrete specified in Eurocode 2 (EN-1992-1-2 2004), of which Fig. 2.12 presents both upper and lower limits of concrete thermal conductivity as a function of temperature. While the upper bound is for siliceous concrete, the lower bound is applicable for carbonate concrete. Besides, Fig. 2.13 shows values of concrete specific heat at three different moisture contents of 0%, 1.5%, and 3% (by weight). For other moisture contents, Eurocode 2 allows linear interpolation. Lastly, concrete density decreases with increasing temperature as shown in Fig. 2.14. This variation is influenced by water loss related to moisture content of concrete.

Fig. 2.12 Variation of concrete thermal conductivity with temperature (EN-1992-1-2 2004)

Fig. 2.13 Variation of concrete specific heat with temperature (EN-1992-1-2 2004)

Fig. 2.14 Variation of concrete density with temperature (EN-1992-1-2 2004)

2.3.1.1.2 Mechanical properties

Compressive strength

Concrete compressive strength versus temperature relationships have been studied since the 1950s (Malhotra 1956, Furamura 1966, Popovics 1973, Anderberg 1976, Harmathy 1993, EN-1992 2004, Purkiss and Li 2013). Generally, concrete compressive strength decreases as temperature increases. The degradation in strength depends on a number of factors such as concrete mix proportion, aggregate type, concrete age, testing condition, etc. Fig. 2.15 shows plots of $f_c(\theta)/f_{ck}(20^{\circ}\text{C})$ for normal-strength concrete specified in Eurocode 2 (EN-1992 2004) as a function of temperature. Here, $f_c(\theta)$ and $f_{ck}(20^{\circ}\text{C})$ denote concrete compressive strengths at $\theta^{\circ}\text{C}$ and at 20°C (ambient), respectively. Fig. 2.15 illustrates that siliceous concrete is more susceptible to temperature than calcareous (carbonate) concrete which is more porous in nature compared to the former. Under high temperature, the high porosity and permeability of carbonate concrete allow water to escape easily, thus reducing formation of pore pressure and microcracks. As a result, carbonate concrete is less susceptible to elevated temperatures compared to siliceous concrete.

Fig. 2.15 Degradation of concrete compressive strength with temperature (EN-1992-1-2 2004)

Tensile strength

There are limited data available on tensile strength of concrete under elevated temperatures, whether based on direct strength or splitting strength (Purkiss and Li 2013). Eurocode 2 (EN-1992 2004) specifies the deterioration of characteristic tensile strength through a reduction factor $k_{c,t}(\theta) = f_{c,t}(\theta)/f_{ct,t}(20^\circ\text{C})$ as given in Fig. 2.16. According to Eurocode 2, the tensile strength of concrete decreases by 50% and 100% compared to its ambient strength as exposed temperature reaches 350°C and 600°C, respectively.

Fig. 2.16 Degradation of concrete tensile strength with temperature (EN-1992-1-2 2004)

Modulus of elasticity

Young's modulus of concrete decreases with increasing temperature. Nonetheless, the level of degradation is dependent on testing methods. Different testing methods, e.g. cylinders for compression strength, dynamic modulus or torsion, result in different values of deterioration. Fig. 2.17 shows percentage reductions in Young's modulus from different test methods.

Fig. 2.17 Variation in Young's modulus for concrete with temperature (Purkiss and Li 2013)

Some simplified equations have been proposed to predict concrete elastic modulus under elevated temperatures. Eq. 2.6 and 2.7 show simplified formulas developed by Li and Purkiss (2005) and Tomasson (1998), respectively. While Eq. 2.6 is specified for normal-strength concrete, Eq. 2.7 is meant for high-performance concrete.

$$E_{ci}(\theta) = \frac{800 - \theta}{740} E_{ci} \leq E_{ci} \quad (2.6)$$

$$E(\theta) = E_{ci}(0.99 + 0.0888\bar{\theta} - 0.20232\bar{\theta}^2 + 0.5279\bar{\theta}^3 - 0.00543\bar{\theta}^4 + 0.000199\bar{\theta}^5) \quad (2.7)$$

for $20^\circ\text{C} < \theta \leq 1000^\circ\text{C}$ with $\bar{\theta} = \frac{\theta}{100}$

where E_{ci} and $E_{ci}(\theta)$ are the concrete modulus of elasticity at ambient and at θ °C, respectively.

Constitutive model (stress-strain relationship)

Typical stress-strain curves of normal-weight concrete under compression specified in Eurocode 2 (EN-1992-1-2 2004) consist of two main portions: an ascending part up to peak stress and a descending part after the maximum stress (Fig. 2.18). Values for $f_{c,\theta}$, $\varepsilon_{c1,\theta}$, and $\varepsilon_{cu1,\theta}$, which are illustrated in Fig. 2.18, are tabulated in Table 3.1 - Eurocode 2 Part 1-2 (EN-1992-1-2 2004).

Range	Stress $\sigma(\theta)$
$\varepsilon \leq \varepsilon_{c1,\theta}$	$\frac{3\varepsilon f_{c,\theta}}{\varepsilon_{c1,\theta} \left(2 + \left(\frac{\varepsilon}{\varepsilon_{c1,\theta}} \right)^3 \right)}$
$\varepsilon_{c1,\theta} < \varepsilon \leq \varepsilon_{cu1,\theta}$	For numerical purposes a descending branch should be adopted. Linear or non-linear model are permitted

Fig. 2.18 Mathematical model for stress-strain relationships of concrete under compression at elevated temperatures (EN-1992-1-2 2004)

Fig. 2.19 shows normalized stress-strain relationships for siliceous concrete at elevated temperatures according to Eurocode 2. As shown in the figure, the peak compressive strength $f_{c,\theta}$ reduces whereas both the strain corresponding to the maximum stress and the ultimate strain increase as temperature increases.

Fig. 2.19 Normalized stress-strain relationships of siliceous concrete at elevated temperature following Eurocode 2

2.3.1.2 Prestressing steel properties under elevated temperatures

Thermal properties of prestressing steel are usually ignored in theoretical and numerical analyses of PCHC slabs as they have very little effect on temperature development over cross sections of the units. This is because the reinforcement ratio in PCHC slabs is relatively low (usually in the range of 0.5% to 1.5%). In addition, heat is transferred through prestressing strands quite rapidly due to high thermal conductivity and low specific heat of steel compared to those of concrete. Thus, in the fire structural analyses of PCHC slabs, strand temperatures are normally assumed to be equal to concrete temperatures at the same level measured from fire exposed soffit. Thus, this section only reviews mechanical and deformation properties of prestressing steel under elevated temperatures.

Stress-strain relationship

There have been several experimental studies on stress-strain relationships of prestressing steels under elevated temperatures (Zheng, Hu et al. 2007, Gálvez, Atienza et al. 2011, Gales, Bisby et al. 2012, Shakya 2016). However, most of the previous tests were conducted on individual wire specimens, and therefore, data obtained from these

studies may not represent the response of strands used in PCHC slabs (Shakya 2016). Thus, the stress-strain-temperature model for prestressing strands specified in Eurocode 2 (EN-1992-1-2 2004) is presented in this section, as it is the most widely accepted model.

The Eurocode stress-strain relationship is defined by five parameters: the slope of the linear elastic range $E_{p,\theta}$, the proportional limit $f_{pp,\theta}$, the maximum stress level $f_{py,\theta}$, the thermal strain $\epsilon_{pt,\theta}$, and the ultimate strain $\epsilon_{pu,\theta}$. They are shown in Fig. 2.20 and given in detail in Table 2.1 and Table 2.2 as a function of temperature.

Fig. 2.20 Eurocode stress-strain relationship of prestressing steel under elevated temperatures (EN-1992-1-2 2004)

Table 2.1 Mathematical model for stress-strain relationships of prestressing steel under elevated temperatures (EN-1992-1-2 2004)

Range	Stress $\sigma(\theta)$	Tangent modulus
$\epsilon_{pp,\theta}$	$\epsilon E_{p,\theta}$	$E_{p,\theta}$
$\epsilon_{pp,\theta} \leq \epsilon \leq \epsilon_{py,\theta}$	$f_{pp,\theta} - c + (b/a)[a^2 - (\epsilon_{py,\theta} - \epsilon)^2]^{0.5}$	$\frac{b(\epsilon_{py,\theta} - \epsilon)}{a[a^2 - (\epsilon - \epsilon_{py,\theta})^2]^{0.5}}$
$\epsilon_{py,\theta} \leq \epsilon \leq \epsilon_{pt,\theta}$	$f_{py,\theta}$	0
$\epsilon_{pt,\theta} \leq \epsilon \leq \epsilon_{pu,\theta}$	$f_{py,\theta}[1 - (\epsilon - \epsilon_{pt,\theta})/(\epsilon_{pu,\theta} - \epsilon_{pt,\theta})]$	-
$\epsilon = \epsilon_{pu,\theta}$	0	-

Parameters	$\varepsilon_{pp,\theta} = f_{pp,\theta} / E_{p,\theta}; \varepsilon_{py,\theta} = 0.02; \varepsilon_{pt,\theta} \text{ \& } \varepsilon_{pu,\theta} \text{ taken as Table 3.2}$
Functions	$a^2 = (\varepsilon_{py,\theta} - \varepsilon_{pp,\theta})(\varepsilon_{py,\theta} - \varepsilon_{pp,\theta} + \frac{c}{E_{p,\theta}})$
	$b^2 = c(\varepsilon_{py,\theta} - \varepsilon_{pp,\theta})E_{p,\theta} + c^2$
	$c = \frac{(f_{py,\theta} - f_{pp,\theta})^2}{(\varepsilon_{py,\theta} - \varepsilon_{pp,\theta})E_{p,\theta} - 2(f_{py,\theta} - f_{pp,\theta})}$

Table 2.2 Values for major parameters of the stress-strain relationships of prestressing steel (cold-worked, Class B) at elevated temperatures (EN-1992-1-2 2004)

Steel temp. θ [°C]	$f_{py,\theta} / (\beta f_{pk})^*$	$f_{pp,\theta} / (\beta f_{pk})^*$	$E_{p,\theta} / E_p$	$\varepsilon_{pt,\theta}$	$\varepsilon_{pu,\theta}$
20	1.00	1.00	1.00	0.050	0.100
100	0.99	0.68	0.98	0.050	0.100
200	0.87	0.51	0.95	0.050	0.100
300	0.70	0.32	0.88	0.055	0.105
400	0.50	0.13	0.81	0.060	0.110
500	0.30	0.07	0.54	0.065	0.115
600	0.14	0.05	0.41	0.070	0.120
700	0.06	0.03	0.10	0.075	0.125
800	0.04	0.02	0.07	0.080	0.130
900	0.02	0.01	0.03	0.085	0.135
1000	0.00	0.00	0.00	0.090	0.140
1100	0.00	0.00	0.00	0.095	0.145
1200	0.00	0.00	0.00	0.100	0.150

* $\beta = 0.9$ for cold worked

Fig. 2.21 shows variations of yield strength and elastic modulus of prestressing steel (strands or wires) as functions of temperature while Fig. 2.22 presents Eurocode 2 stress-strain curves of cold-worked (Class B) prestressing steel as a function of temperature. Basically, both yield strength and elastic modulus of prestressing steel reduce with increasing temperature.

Fig. 2.21 Variation of yield strength and elastic modulus of Class B cold-worked prestressing steel (wires and strands) with temperature based on Eurocode 2

Fig. 2.22 Stress-strain relationship of Class B cold-worked prestressing steel (wires and strands) at elevated temperatures based on Eurocode 2

Thermal elongation of prestressing steel

The variation of thermal elongation in prestressing steel with temperature is specified in Eurocode 2 (EN-1992-1-2 2004) and shown in Fig. 2.23. Thermal strain ($\Delta L/L$) in Fig.

2.23 is measured on un-loaded specimens under heating with respect to original length at 20°C.

Fig. 2.23 Total thermal elongation of prestressing steel (EN-1992-1-2 2004)

Thermal properties of concrete and prestressing steel significantly affect structural behavior of PCHC slabs. The next section reviews possible failure modes of PCHC slabs when subjected to fire.

2.3.2 Failure modes of PCHC slabs under fire conditions

Under fire conditions, the load bearing capacity of a PCHC slab decreases due to degradation of material properties and temperature-induced stresses (Fellinger 2005). If the bearing capacity falls below the imposed moment or shear force, failure occurs.

Previous studies have shown that PCHC slabs exposed to fire can fail in bending, anchorage, shear-compression, and web-shear modes (Borgogno and Fontana 2000, Fellinger 2005, Chang, Moss et al. 2010, Aguado, Espinos et al. 2012, Torić, Divić et al. 2012, Jansze, Klein-Holte et al. 2014, Shakya and Kodur 2015). Among the four, fire tests conducted over the past 40 years showed that shear and anchorage failures dominate the fire behavior of PCHC slabs rather than bending failure (Fellinger 2004).

Besides the four common failure modes, fire-induced spalling is another potential failure observed in concrete members subjected to fire. Spalling is critical because once it occurs, prestressing strands are directly exposed to high temperatures, resulting in a quick reduction in strength and stiffness. Spalling commonly occurs in a concrete temperature range of 200 to 500°C. Studies have shown that susceptibility to spalling of concrete is a function of many factors such as permeability, moisture content, size of section, concrete tensile strength, heating rate, type of aggregate, load intensity, and loading type (Hertz 2003, Hertz and Sørensen 2005, Kelly and Purkiss (2008), Zheng, Hou et al. 2010). Since spalling is controlled by many factors, it is not easy to predict. However, it is widely accepted that high-strength concrete is more vulnerable to spalling compared to normal-strength concrete. This is because high-strength concrete is less porous compared to normal-strength concrete. The low porosity of high-strength concrete results in slower diffusion of water vapor, causing higher pore pressure and thus promoting spalling to occur.

2.3.3 Review of previous research works on PCHC slabs in fire

In the last four decades, many studies have been conducted on fire-exposed PCHC slabs. This section reviews the most relevant studies on structural fire performance of PCHC slabs to identify technical gaps in this area. The review is categorized into three main groups, viz. experimental investigations, finite-element based and analytical approaches. Primary parameters and findings of each study are discussed; limitations and drawbacks of those are stated, followed by identified shortcomings.

2.3.3.1 Experimental investigations

Experimental studies discussed in this section are summarized in Table 2.3 while detailed information of such studies is reviewed subsequently.

Table 2.3 Summary of reviewed experimental studies on PCHC slabs exposed to fire

Research	Slab depth (mm)	Mode of failure	Parameters
Andersen and Lauridsen (1999)	185 to 270	Bonding and shear	Slab depth
Borgogno and Fontana (2000)	160 to 240	Anchorage, web-shear, flexural-shear, punching shear	Supporting conditions (flexural and rigid supports)
Fellinger (2005)	200, 260, 265, 400	Shear and anchorage	Aggregate type, load level, and restraint condition
Bailey and Lennon (2008)	200	Flexural	Restraint condition
Aguado <i>et al.</i> (2012)	250	Flexural	Axis distance and prestressing type
Shakya and Kodur (2015)	200	Flexural	Aggregate type, load level, fire exposure, restraint condition

Fire resistances of PCHC slabs (Andersen and Lauridsen 1999)

Andersen and Lauridsen (1999) carried out three fire tests on six PCHC units (each test included two identical units) to determine fire resistance of the test specimens, and to use the test results to evaluate four different methods of estimating fire rating. The slabs were 1.2 m wide, 6.35 m long with depth ranging from 185 to 270 mm and were subjected to the standard fire curve ISO 834. Based on calculations using the four methods, flexural failure would be expected. However, bonding failure causing premature shear failure was observed in all the test specimens. It showed that conventional calculations did not accurately predict the failure mode in the tests. Moreover, despite the fact that all the hollow-core slabs were approved to sustain 60-min fire resistance, their resistance was

within 26 min of fire exposure. The authors raised some concerns about fire resistance of the test specimens, as well as the validity of conventional fire-resistance-prediction methods.

Slim floor slabs with different support conditions (Borgogno and Fontana 2000)

Borgogno and Fontana (2000) conducted six fire tests on PCHC slabs with and without topping and collected data from another ten fire tests from literature to investigate the effect of different supporting conditions on structural behavior of slim floor slabs. Investigated supporting conditions included flexible and rigid supports, as well as supporting lengths. All slabs were 1.2 m in width with different lengths and the slab depth ranged from 160 to 240 mm. Anchorage, web-shear, flexural-shear, and punching shear failures were four failure modes observed in the tests. The study showed that, in the case of fire, thermal stresses developed in concrete and reductions in material strength and stiffness took place and governed the structural behavior of the slim floor system. Results from the research also demonstrated that the fire behavior of hollow-core elements was influenced by support conditions. More specifically, the use of flexible supports (through the use of steel beams to support PCHC slabs) resulted in lower fire resistance compared to the use of rigid supports. This is because under elevated temperatures, deflection of the steel beams induced additional torsional moments and uneven shear loads in hollow-core slabs, causing early failure.

Shear and anchorage behavior (Fellinger 2005)

Fellinger (2005) collected data from 80 fire tests in literature and showed that shear failure dominated in approximate 25% of all the tests. In addition, the failure was typically premature and occurred before achieving the slabs' required fire resistance. Distinctions between different shear failure modes (e.g. shear-compression, shear-tension and anchorage failure) could not be made. This was partly because these tests were not reported properly as they were conducted to show satisfactory fire resistance instead of obtaining scientific insight (Fellinger 2005). Thus, to gain an overall

understanding of shear and anchorage behavior of fire-exposed PCHC slabs and to develop finite-element models to predict such behavior, Fellingner (2005) carried out standard fire tests on 25 PCHC units. A total of 21 (out of 25) fire tests were conducted on double-rib specimens to observe thermal cracking from two sides of slab webs (Fig. 2.24). All the hollow-core units in the program were 4 m long, simply supported, and were manufactured by either the extrusion process or slip form method. Four different slab depths, i.e. 200, 260, 265, and 400 mm, were employed. The effect of slab thickness, axis distance, load level, and support conditions (restrained or unrestrained), on structural fire behavior of hollow-core elements, was evaluated. Besides conducting structural fire tests, the author carried out 60 small-scale material tests to calibrate proposed concrete constitutive models at elevated temperatures (up to 600°C). The structural fire tests showed that premature failures (shear) occurred in deeper units, e.g. 265 mm thick or 400 mm thick slabs. Those slabs failed in a range of 24 to 60 min of fire exposure. The study concluded that incompatible thermal expansion led to cracking and slip of strands within the first 15 min of standard fire exposure. Consequently, the shear and anchorage capacities quickly decreased in the early stage of fire exposure, eventually resulting in premature collapse. Such failures should be considered in the fire design of PCHC slabs (Fellinger 2005). Furthermore, aggregate type, load level, and restraint condition had significant effects on structural fire behavior of PCHC slabs. In particular, hollow-core slabs with carbonate aggregate exhibited better performance under elevated temperatures compared to those using siliceous aggregate. In addition, fire-exposed slabs with higher load level failed earlier compared to those under lower load level. Finally, axial restraint significantly benefited structural performance of PCHC slabs under fire conditions.

Fig. 2.24 Thermal cracks in the web during fire tests (Fellinger 2005)

Large-scale fire tests (Bailey and Lennon 2008)

Bailey and Lennon (2008) conducted two large-scale fire tests during 3 h on a total of 15 hollow-core slabs, supported on protected steelwork to evaluate fire performance of hollow-core floor systems. All hollow-core units were 1.2 m wide, 7 m long, 200 mm thick, and were subjected to a very severe compartment fire (Fig. 2.26). Fire load was achieved using standard wooden cribs whereas an applied load of 4.5 kN/m^2 was applied using distributed sandbags (Fig. 2.25). The two fire tests were identical in terms of specimens used, test setup, loading scheme, etc. Nonetheless, there was a difference in the connection details between the hollow-core units and the supporting steel beams. The study showed that the hollow-core floors behaved very well under fire conditions. In addition, different end restraint conditions did not affect vertical displacement of the hollow-core systems. However, a lateral compressive strip formed at the slab end due to some degrees of restraint to thermal expansion enhanced both flexural and shear capacities.

Fig. 2.25 Loading scheme (Bailey and Lennon 2008)

Fig. 2.26 Two full-scale fire tests on PCHC slabs (Bailey and Lennon 2008)

Flexural behavior of fire-exposed PCHC slabs (Aguado, Espinos et al. 2012)

Aguado, Espinos et al. (2012) conducted four full-scale fire tests to study the effect of reinforcement arrangement on flexural behavior of PCHC slabs. All slabs were 1.2 m wide, 5.7 m long, and 250 mm thick, characterized by noncircular voids. The influence of reinforcement arrangement was studied through two major parameters: axis distance (varying from 30.1 to 46.7 mm) and prestressing strand type (5 mm diameter wire and 9.5 mm diameter strand). All the slabs were loaded using a four-point loading scheme and were subjected to the standard fire curve ISO 834. Four types of crack, i.e. thermal, flexural, splitting, and longitudinal cracks, were observed in the tests (Fig. 2.27). The study concluded that while thermal cracking occurred at the first 20 min of fire exposure

and governed deflection behavior of the slabs during this stage, the slabs failed at the last stage (after 65 min of heating) due to the loss in strength of prestressing steels. In addition, prestressing wires showed better performance compared to strands as the wires did not cause splitting to adjacent concrete (Fig. 2.27(d)).

Fig. 2.27 Crack types observed in fire tests (Aguado, Espinos et al. 2012)

Response of high-strength PCHC slabs (Shakya and Kodur 2015)

Shakya and Kodur (2015) tested six high-strength PCHC slabs to study responses of hollow-core slabs under fire conditions. All slabs were 4 m in length, 1.2 m in width, and 200 mm in depth, characterized by six circular cores. Concrete compressive strength was in the range of 74 MPa (carbonate concrete) to 91 MPa (siliceous concrete). The specimens were tested under a 4-point loading scheme and were subjected to three different fire scenarios. Effects of aggregate type, load level, different fire exposure and restraint conditions were studied. Observations from the tests showed that all the PCHC slabs sustained service loads for at least 2 h under standard and designed fire scenarios. The study concluded that fire scenario and load level had substantial influences on fire performance of the PCHC slabs. Furthermore, the presence of axial restraint enhanced

fire resistance of the slabs by an additional 30 min. Finally, carbonate concrete exhibited higher fire resistance compared to siliceous concrete.

Comments on the literature of experimental studies

The experimental studies showed that shear and anchorage failures were more critical for PCHC slabs compared to flexural failure. Fire-exposed PCHC slabs usually exhibited satisfactory performance in flexural fire tests while premature shear and anchorage failures frequently occurred at an early stage of fire exposure (within 60 min). However, there are still limited fire shear tests on PCHC slabs, especially for slabs with depths more than 300 mm. Most of the fire tests in the literature were conducted on PCHC slabs with thickness ranging from 150 mm to 265 mm. No fire tests performed on 320 nor 500 mm thick slabs could be found and only 4 fire tests were conducted on 400 mm thick, double-rib hollow-core units sawn from a standard hollow-core unit of 1.2 m wide. As a result, fire tests from the literature did not provide adequate information for an in-depth study on shear response of PCHC slabs subjected to fire. Consequently, the way fire-exposed PCHC slabs fail in web-shear has not been well established.

2.3.3.2 Numerical studies

Fire tests are not only complex but also highly demanding in terms of time, safety, and resources. Consequently, it is not practical to conduct many fire tests to investigate every possible parameter. In such a case, numerical approach serves as an alternative solution to assessing fire performance of structures. Over the past twenty years, there have been many numerical studies on structural performances of PCHC slabs in fire. Notable studies are briefly reviewed in this section.

Temperature development vs fire resistance (Dotreppe and Franssen 2004)

Dotreppe and Franssen (2004) carried out numerical studies on PCHC slabs exposed to the fire curve ISO 834 using SAFIR software (Franssen, Kodur et al. 2004). Effect of hollow-core geometry (cavities) on temperature and transverse cracking developments, and effect of lateral restraints on deflection of PCHC slabs, were studied. Numerical

results demonstrated that thermal stresses were critical during the first 30 min of fire exposure, and cavities significantly affected the temperature evolution, as well as subsequent fire resistance of PCHC slabs. Besides, lateral restraints enhanced the fire performance of the PCHC units.

Shear and anchorage failures (Fellinger 2005)

Fellinger (2005) developed a numerical model using FE package DIANA (DIANA 2000) to study shear and anchorage mechanisms of fire-exposed PCHC slabs. The FE model incorporated new constitutive models for concrete and bond interface at elevated temperatures developed by the same author. Furthermore, splitting cracks, vertical cracks and slip development in fire-exposed PCHC slabs were carefully considered in the numerical model (Fig. 2.28). The FE model was validated against 25 full-scale fire tests on PCHC slabs subjected to shear (Fellinger 2005). The variations of tensile strength of concrete, fracture energy in both compression and tension, and concrete thermal expansion on fire behavior, were investigated. The study concluded that fire resistance of PCHC slabs was strongly affected by concrete thermal expansion, fracture energy and lateral restraint conditions.

Fig. 2.28 Overview of the FE model (Fellinger 2004)

Hollow-core floor system (Chang, Moss et al. 2010)

Chang, Moss et al. (2010) employed SAFIR (Franssen, Kodur et al. 2004) to investigate the effect of “system” (considering several units together rather than analyzing only one unit), aspect ratio (floor length/floor width), and types of side support on fire behavior of PCHC floor systems. Beam grillage and shell elements were used to model hollow-core units and concrete topping, respectively. Constitutive model in Eurocode 2 was incorporated in this simulation. Flexural behavior was simulated, considering the effects of prestressing, thermal strains, mechanical stresses, and continuity between the hollow-core units. The study showed that for hollow-core floor systems, it was not suitable to use tabulated data (prescriptive method) for two-way solid slabs presented in Table 5.8 in Eurocode 2 Part 1-2. Such a table was too optimistic about the effect of side beams when the aspect ratio was greater than 1:1.5.

Fig. 2.29 (a) Hollow-core system (b) Modeling in FE (Chang, Moss et al. 2010)

Advanced analysis of PCHC slabs to different fires (Ellobody 2014)

A nonlinear 3-D FE model for analyzing of PCHC slabs at elevated temperatures was developed by Ellobody (2014) using ABAQUS (Simulia 2011). Nonlinearities of mechanical and thermal material properties, bond interface between concrete and prestressing steels, and thermal interaction in cavities of PCHC slabs, were carefully incorporated in the FE model. The effects of slab geometry, hollow-core shape, load ratio, and fire scenario were examined in the numerical analyses. The research showed that failures occurred in the investigated slabs due to tensile splitting and longitudinal

cracks developed between 15 to 25 min of fire exposure. No shear failure was observed, and fire endurance of PCHC slabs was primarily dependent on tensile splitting failure.

Factors governing shear response of PCHC slabs (Kodur and Shakya 2017)

Kodur and Shakya (2017) developed a numerical model to study fire responses of PCHC slabs using ANSYS (ANSYS 2014). The FE model considered temperature-induced reduction in concrete and prestressing strands. William and Warnke's constitutive models were adopted in the FE model to represent compressive and tensile behavior of concrete. The model was validated against two flexural tests on 200 mm thick slabs from Shakya and Kodur (2015) and one shear test of a 265 mm thick slab from Jansze, Acker et al. (2012). Variations in the parametric study included slab depth, level of prestressing, axial restraint, load level, loading pattern, and fire scenario. Results from the numerical study showed that such parameters had significant effect on fire response of PCHC slabs.

Comments on the literature of numerical studies

The literature review shows that there have been many finite element based studies on structural responses of PCHC slabs under fire conditions. Nonetheless, due to the paucity of experimental results on deep PCHC slabs, most of the developed FE models were validated using fire tests on shallow PCHC slabs. However, behavior of the two can be greatly different even under identical loading and heating schemes. For instance, with the same shear span of 1000 mm, a 400 mm thick slab can fail by web-shear while a 200 mm thick slab can collapse by flexure. Studies on deep PCHC slabs cannot rely solely on FE models which were verified by fire tests on shallow units. Therefore, an independent numerical model validated using deep PCHC slabs is needed to accurately predict response of the slabs under fire.

2.3.3.3 Evaluating shear capacity of fire-exposed PCHC slabs – Analytical approach

Despite the fact that shear or anchorage failure can dominate behavior of PCHC slabs under fire conditions rather than flexural failure, only until 2011, an empirical formula

for shear capacity of single span PCHC slabs subjected to fire was given in a design code (EN 1168:2005+A3:2011 Annex G (EN-1168:2005+A3 2011)). It is worth to note that before 2011, only flexural failure was considered in all code provisions to evaluate fire resistance of concrete members. Besides the deterioration in strengths of concrete and prestressing steels, the shear formula in EN 1168 directly incorporates the reduction in bond strength of anchorage at elevated temperatures.

Shear resistance according to EN 1168:2005+A3:2011

In EN 1168, shear capacity under fire conditions is given as follows:

$$V_{Rd,c,fi} = [C_{\theta,1} + \alpha_k C_{\theta,2}] b_w d \quad (2.8)$$

where

$$C_{\theta,1} = 0.15 \min (k_{p(\theta_p)} \sigma_{cp,20^{\circ}C}; \frac{F_{R,a,fi,p}}{A_c})$$

$$\alpha_k = 1 + \sqrt{\frac{200}{d}} \leq 2$$

$$C_{\theta,2} = \sqrt[3]{0.58 \frac{F_{R,a,fi}}{f_{yk} b_w d} f_{c,fi,m}}$$

$V_{Rd,c,fi}$ is fire shear strength in regions cracked in bending (N);

$C_{\theta,1}$ is coefficient accounting for concrete stress under fire conditions;

$k_{p(\theta_p)}$ is strength reduction factor for the prestressing steel in Eurocode 2 Part 1-2;

$\sigma_{cp,20^{\circ}C}$ is average concrete stress due to prestressing at room temperature;

A_c is concrete area in (mm²)

$F_{R,a,fi,p}$ is force capacity of prestressing steel anchored in considered cross section;

$C_{\theta,2}$ is coefficient accounting for anchored longitudinal reinforcement;

$F_{R,a,fi,p}$ is force capacity of prestressing steel anchored in considered cross section;

f_{yk} is characteristic yield strength of the reinforcement;

$f_{c,fi,m}$ is average strength of concrete at elevated temperature;

b_w is total web thickness of hollow core slab;

d is effective depth of hollow core slab at ambient temperature.

The presented empirical formula in EN 1168 exhibits a potential use for calculating web-shear strength of fire-exposed PCHC slabs since the model takes into account sections near bearing regions which are uncracked by bending. In addition, EN 1168 approach considers the reduction in bond strength of anchorage at elevated temperatures, which significantly affects web-shear behavior.

Comments on the approach

Jansze, Acker et al. (2012) collected data from 42 shear tests conducted between 1966 and 2010 in Europe to validate the shear formula in EN 1168. The validation showed that the shear capacity obtained from the 42 shear tests was on average 7% higher than the shear capacity predicted by Annex G of EN 1168. From their findings (Jansze, Acker et al. 2012), they concluded that the EN 1168 equation was conservative and valid for all hollow-core slab depths up to 400 mm. Having said that, it should be noted that EN 1168 does not account for the unfavorable effect of temperature-induced stress in its calculations. This may lead to unconservative predictions for web-shear capacity of deep PCHC slabs subjected to fire because thermal stress induced by thermal gradients are more pronounced for deep units. Thus, more test data on deep PCHC slabs are needed to verify the formula.

2.4 Review on the use of fibers for enhancing shear capacity of PCHC slabs

Since stirrups cannot be implemented in PCHC slabs and shear failure is critical, a number of shear-strengthening techniques have been investigated, e.g. applying core filling (Palmer and Schultz 2011), using fiber-reinforced polymer composite sheets (Fig. 2.30) (Wu, El Ragaby et al. 2016), or using polypropylene (PP) or steel fibers (Peaston and Paine 1999, Palmer and Schultz 2011, Cuenca and Serna 2013, Simasathien and Chao 2015, Dudnik, Milliman et al. 2017) in the slabs. Among them, the last one has attracted the most attention.

Fig. 2.30 Shear strengthening technique using FRP (Wu, El Ragaby et al. 2016)

Peaston and Paine (1999) were the pioneers of using hooked steel fibers in extruded PCHC slabs. Thereafter, Cuenca and Serna (2013), Simasathien and Chao (2015), and Dudnik, Milliman et al. (2017) also investigated the use of hooked steel (H) fibers in hollow-core slabs. It was consistently observed that the addition of steel fibers increased both the shear strength and ductility of the hollow-core slabs. The extent of improvement in shear capacity depended on shear-span-to-effective-depth (a/d) ratio. It is shown that considerable increases in shear strength were observed in tested slabs with a/d ranging from 2.0 to 2.8 while there were only minor increases in shear capacity of specimens with a/d greater than 3.0. Moreover, due to the manufacturing process, steel fibers in PCHC slabs were prone to be aligned vertically in the concrete webs. Therefore, they were more effective in hollow-core slabs compared to in beams. However, while Cuenca and Serna (2013) and Peaston and Paine (1999) reported that no major issues were observed in production process with the use of hooked steel fibers up to 0.89 and 1% in volume fraction in very dry concrete, Dudnik, Milliman et al. (2017) observed compaction problems when concrete with a fiber volume content of 0.76% was cast. Poor compaction would result in reduced shear resistance. Besides the investigations on PCHC slabs incorporating steel fibers, Palmer and Schultz (2011) studied the use of PP fibers on extruded hollow-core slabs. With a volumetric fiber content of 0.67 and 1.0%, shear strength increased by as much as 20 and 40%, respectively, compared to that of a companion slab without fibers. The study showed a potential use of PP fibers in PCHC slabs to enhance shear resistance.

The literature review shows that previous studies were limited to the use of hooked steel fibers while other types of steel fiber have not been investigated. In addition, the upper limit of hooked steel fibers used in nearly zero-slump concrete in the extrusion process has not been quantified. Moreover, although the use of PP fibers enhancing shear capacity of PCHC slabs showed promising results, there is only one study reported to date. Therefore, more experimental investigations on the use of fibers including steel and PP fibers in hollow-core slabs are needed.

Furthermore, it is shown that there have been several studies on fibers enhancing the shear resistance of PCHC slabs at room temperature. However, to date, no experimental studies at structural level have been carried out on fiber-reinforced-concrete (FRC) slabs under fire conditions. Therefore, the effectiveness of PP and steel fibers on structural performance of FRC members under elevated temperatures has not been understood or quantified.

2.5 Summary

This chapter reviews shear studies on PCHC slabs at ambient and under fire conditions, as well as techniques to improve shear capacity of the slabs. Concerning web-shear resistance of PCHC slabs, the following major shortcomings in the existing research are identified:

- (1) Web-shear failure is more critical to PCHC slabs with depth exceeding 300 mm with flat webs compared to shallow slabs characterized by circular voids. However, mechanisms of which deep PCHC slabs are more susceptible to web-shear failure have not been well understood. Furthermore, in many cases, design codes are still unconservative in predicting shear strength of deep PCHC slabs. Thus, further investigations on deep PCHC slabs as well as a more rational approach for assessing shear capacity of hollow-core slabs are needed.
- (2) In the event of fire, the web-shear capacity of PCHC slabs significantly decreases due to temperature-induced degradation in strength of concrete and prestressing

steel and induced tensile stress in concrete webs which is more pronounced for deeper units. However, most of fire shear tests in the literature were conducted on shallow PCHC slabs. There are only 4 fire tests on PCHC slabs with depth of 400 mm. As a result, these fire tests did not provide adequate information for an in-depth study on shear response of PCHC slabs subjected to fire. Consequently, the way fire-exposed PCHC slabs fail in web-shear has not been well established. Further experimental and numerical investigations are necessary to bridge this gap.

- (3) Fire shear tests from the literature showed that hollow-core slabs commonly failed in web-shear mode within 60 min of fire exposure whereas they were designed to sustain at least 1 h fire rating. It is believed that thermal stress plays an important role in behavior of PCHC slabs subjected to shear. However, it has not been fully quantified.
- (4) There are limited test data on the use of steel and PP fibers to enhance shear capacity of PCHC slabs. Moreover, the use of steel fibers has been limited to hooked-steel type while other types of fibers have not been investigated. Furthermore, the upper limit of hooked steel fibers to be used in nearly zero-slump concrete in the extrusion process has not been quantified.
- (5) Although the use of fibers enhancing shear capacity of concrete members has been extensively investigated at ambient condition, it has not been quantified under fire conditions yet.

From the identified knowledge gaps, research objectives and scope of work as presented in Chapter 1 (Section 1.3) were formed. To achieve these objectives, an experimental program consisting of 21 deep PCHC slabs were proposed and conducted, of which 9 specimens were to investigate shear behavior of conventional PCHC slabs, while the other 12 were to examine shear responses of FRC hollow-core slabs. The hollow-core units were tested at both ambient and fire conditions. The next chapter (Chapter 3) presents an overview of the testing program. Properties of the test specimens, test rig, instrumentation as well as heating conditions are also given.

CHAPTER 3 EXPERIMENTAL PROGRAM ON DEEP PCHC SLABS WITH AND WITHOUT FIBERS

3.1 Introduction

To address the technical gaps in the existing research identified in Chapter 2, an experimental program consisting of 21 deep PCHC slabs, divided into 4 series as shown in Fig. 1.5, was proposed and conducted at ambient and under fire conditions at Nanyang Technological University (NTU), Singapore. This chapter provides an overview of the test program. Specimen design, detailed material properties, test setup, instrumentation, as well as heating conditions are introduced. The main objectives of the experiments are to investigate web-shear behavior of PCHC slabs with and without fibers at ambient and under fire conditions. While the first two series focus on shear behavior of conventional PCHC slabs, the last two series investigate shear performance of fiber-reinforced (FR) PCHC slabs. The experiments were conducted in sequence, from Series I to Series IV. Linkages between them are shown in Fig. 3.1.

Fig. 3.1 Relationships between the four test series

It should be noted that except for H.HC.400 (a heat transfer test in Series II as shown in Table 3.2), Series I and II specimens with the same depth were cast in one batch. Similarly, specimens in Series III and IV with identical fiber type and content were cast in the same batch. As the mix design for each batch was fixed, the concrete strength values defined by material tests in Series I and III were used for the twin specimens in Series II and Series IV, respectively. The experimental program is introduced in two main parts, i.e. tests conducted on conventional PCHC slabs (Series I & II) and tests on FRC slabs (Series III and IV).

3.2 Experimental program on conventional PCHC slabs (Series I & II)

3.2.1 Overview

Full-scale shear tests on PCHC specimens were conducted at ambient (Series I) and under fire conditions (Series II), aiming to:

- 1) Gain an in-depth understanding of web-shear behavior of PCHC slabs at room and elevated temperatures.
- 2) Provide experimental data for verifying FE models to simulate web-shear response of PCHC slabs at ambient and under fire conditions. The models, in turn, extend understanding shear behavior of the slabs as well as to enlarge the existing experimental database for PCHC slabs.
- 3) Provide test data for evaluating shear formulas in some design codes such as ACI 318-14 (ACI 2014), Eurocode 2 (EN-1992-1-1 2004), and EN 1168 (EN-1168:2005+A3 2011).

A summary of the test programs of Series I and II is given in Table 3.1 and Table 3.2, respectively. A total of four shear tests were conducted in Series I while another five fire tests were performed in Series II.

Table 3.1 Summary of test program (Series I: At ambient temperature)

Slab ID.	Slab thickness (mm)	Shear span (mm)	Shear-span-to-effective- depth ratio (a/d)	Failure load (kN)
HC1.320	320	800	2.86	184
HC2.400	400	800	2.25	511
HC3.500	500	800	1.79	1162
HC4.320	320	600	2.25	194

Table 3.2 Summary of test program (Series II: Under fire conditions)

Slab ID.	Slab thickness (mm)	Shear span (mm)	Fire scenario	Test load
H.HC.400	400	n.a	ISO 834	n.a
F.HC1.320	320	800	Non-standard fire	40% of HC1.320
F.HC2.400	400	800	Non-standard fire	40% of HC2.400
F.HC3.500	500	800	Non-standard fire	40% of HC3.500
F.HC4.500	320	600	Non-standard fire	25% of HC3.500

In Series I, full-scale shear tests were performed on four deep PCHC slabs designated as HC1.320, HC2.400, HC3.500, and HC4.320, in which 320, 400, and 500 represented slab thickness (Table 3.1). It should be noted that the original plan for Series I consisted of only three tests on slabs with different depths (HC1.320, HC2.400, and HC3.500). However, as HC1.320 failed in flexural-shear instead of web-shear as predicted by ACI 318-14 (ACI 2014), one additional shear test was conducted on another 320 mm thick slab (named as HC4.320) with a shorter shear-span-to-effective-depth ratio (a/d) to generate an additional test result for this slab depth.

With the exception of H.HC.400 (heat transfer test), all the specimens in Series II were designated as F.HCX.Y, of which F, X and Y referred to testing under fire conditions, the order of testing, and overall slab thickness, respectively. It is also noteworthy that the

original test plan for Series II included four full-scale fire shear tests to investigate shear failure mechanism of deep PCHC slabs subjected to fire and to obtain experimental data for numerical and analytical verifications. However, as hollow-core slabs commonly failed at an early stage in fire shear tests (within 60 min of fire exposure) as reported in the literature (Andersen and Lauridsen 1999, Fellingner 2005, Jansze, Acker et al. 2012), there was no temperature data up to 3 hours to verify a developed FE heat transfer model. Thus, one independent heat transfer test (H.HC.400) was conducted on another 400 mm thick slab to obtain temperature profile up to 3 hours. Therefore, there were a total of five fire tests including four structural shear tests and one heat transfer test conducted in Series II.

It is also worth mentioning that due to the large dimensions of the full-scale structural tests in Series II (4.15 m long $\times$ 1.2 m wide) and limitations of an existing electrical heating furnace, the standard fire ISO 834 could not be simulated. Thus, the four structural fire tests in Series II (F.HC1.320, F.HC2.400, F.HC3.500, and F.HC4.500) adopted a NON-ISO heating curve instead of ISO 834 fire. Fig. 3.2 shows the electric furnace and the heating curve used in the structural fire tests. As can be seen, the adopted curve was able to reach 1000 °C at 60 min of fire exposure with a heating rate of 16 °C/min and to remain at this temperature until the end of the tests. In contrast, a smaller furnace (which was able to simulate ISO 834 fire) was used to conduct the heat transfer test for H.HC.400. This is because, in general, no loads are needed in heat transfer tests. Thus, a 2.2 m long by 1.2 m wide H.HC.400 was selected to fit into the ISO furnace in NTU Annex Lab so that the standard ISO 834 fire curve could be applied. From temperature profiles acquired from the heat transfer test under ISO 834 fire together with those in the structural fire tests using the parametric curve, it would be sufficient to validate the FE heat transfer model with both NON-ISO and ISO fire.

Fig. 3.2 Electrical furnace and heating curve employed in the structural fire tests

3.2.2 Test specimens of Series I and II

3.2.2.1 Geometric properties

Since the objective of this study was on web-shear behavior of deep PCHC slabs, specimens with thickness of more than 300 mm, i.e. 320, 400 and 500 mm and characterized by non-circular voids were employed. These sections are commonly adopted for deep PCHC slabs. Except for H.HC.400 of the heat transfer test (its geometric and material properties will separately be introduced in Chapter 5), a specimen length of 4.15 m was selected to be in compliance with the specimen-length requirement for fire tests in EN 1168:2005+A3:2011 (EN-1168:2005+A3 2011) while a standard width of 1.2 m was chosen for all the specimens. All the slabs were fabricated by a local precast company using the extrusion method (Fig. 3.3). Fig. 3.4 shows the specimen cross sections while Table 3.3 presents the detailed geometric properties.

Fig. 3.3 Casting hollow-core specimens using the extrusion method

Fig. 3.4 Nominal cross sections of specimens

Table 3.3 Geometric properties of PCHC specimens

Slab ID.	320 mm thick slab	400 mm thick slab	500 mm thick slab
Cross-section area, A (m ²)	0.187	0.213	0.259
Cross-section depth, h (mm)	320	400	500
Cross-section width, b (m)	1.2	1.2	1.2
Specimen length, L (m)	4.15	4.15	4.15
Total web width, b_w (mm)	240	266	292
Centroid from bottom slab, y_b (mm)	159	198	249
2 nd moment of area, I (m ⁴)	2.47×10^{-3}	4.42×10^{-3}	8.22×10^{-3}
1 st moment of area top part, S_t (m ³)	1.53×10^{-2}	2.19×10^{-2}	3.26×10^{-2}
Self weight of slab, w (kN/m ²)	3.78	4.31	5.23
No. of strands/ strand diameter (mm)	5/9.6	2/9.6 & 9/12.9	10/12.9
Axis distance (mm)	40	45	52.5
Prestressing steel ratio (%)	0.147	0.470	0.382

3.2.2.2 Material properties

Prestressing steel

There were two types of prestressing steel used in the slabs: 9.6 mm diameter strands with a nominal cross-sectional area of 55 mm² and 12.9 mm diameter strands with an area of 99 mm² (Table 3.3). All prestressing strands were seven-wire, low-relaxation type with a nominal tensile capacity of 1860 MPa. Material tests for prestressing strands were conducted on strand samples with a length of about 1000 mm by the precast fabricator. While Table 3.4 shows measured strand properties, stress-strain curves of the two types of strand used are also included in Fig. 3.5.

Table 3.4 Measured strand properties

Strand type	Number of samples	Mean sample length (mm)	Mean sectional area (mm ²)	Mean tensile strength (MPa)	Mean Elastic Modulus (GPa)
Ø9.6 mm	3	1004	55.6	1965	199.7
Ø12.9 mm	3	1007	99.9	1918	202.7

Fig. 3.5 Stress-strain curves of two types of strand used

Before casting the concrete, the strands were stressed to 62% of their nominal ultimate strength, corresponding to a jacking stress of 1150 MPa for all specimens. Nonetheless, the prestressing level in the strands decreased with time due to prestress loss. For pre-tensioned concrete elements, a total loss of prestressing force is attributed to the cumulative contribution of losses from concrete elastic shortening, relaxation (or creep) of strands, shrinkage and creep of concrete. Total prestressing loss of each specimen up to the testing date was calculated following Eurocode 2 (EN-1992 2004) and is presented in Table 3.5. Such losses will be incorporated in the theoretical and numerical shear-strength predictions in the following chapters (Chapters 4 to 8).

Table 3.5 Calculated prestress losses based on Eurocode 2

Slab ID.	Concrete age at testing (days)	Loss of prestress (kN) due to				Total loss (kN)	Total loss (%)
		Elastic shortening	Relaxation of strands	Shrinkage of concrete	Creep of concrete		
HC1.320	460	2.6	6.3	16.1	2.3	27.3	8.6
HC2.400	400	30.6	23.3	59.1	31.7	144.7	11.4
HC3.500	408	24.5	23.1	58.5	25.4	131.4	12.4
HC4.320	487	2.6	6.3	16.1	2.2	27.2	8.6

Concrete

The nine specimens in Series I & II were designed with a characteristic strength of 40 MPa. Concrete compressive strength at 28 days was measured on 150 mm cube specimens at the precast plant and was converted to cylinder strength using Table 3.1 – Eurocode 2 (EN-1992 2004). Table 3.6 presents concrete-strength values obtained from the compression tests. The results show that the hollow-core specimens met the design strength requirement and were suitable for testing.

Table 3.6 Concrete compressive strength at 28 days

Slab	Number of cube specimens	Average concrete cube strength at 28 days (MPa)	CoV (%)	Corresponding concrete cylinder strength (MPa)
HC1.320	3	65.3	0.6	54
HC2.400	3	65.2	0.6	54
HC3.500	3	65.7	0.6	54

Since concrete cubes (or cylinders) were not available on the test day, material tests were conducted on cylindrical cores obtained from the specimens in Series I by coring method. First, concrete at undamaged parts of the already tested specimens was hacked into small pieces. Thereafter, a coring machine with a core diameter of 50 mm was employed for sample taking. The concrete cores were then cut into $\varnothing 50$ mm by 100 mm long

cylindrical cores for testing. Fig. 3.6 shows the process of obtaining cylindrical samples for the material tests.

Fig. 3.6 Process of getting cylindrical cores

In studies of shear behavior, concrete tensile strength is important. Therefore, in the material tests, not only compressive strength tests (Fig. 3.7(a)), but also splitting tensile strength tests (Fig. 3.7(b)) were conducted. It is noteworthy that the standard dimensions of concrete cylinders for compressive strength test are $\varnothing 100 \text{ mm} \times 200 \text{ mm}$ or $\varnothing 150 \text{ mm} \times 300 \text{ mm}$. However, due to hollow-core slab configurations, the maximum sample size that coring could achieve was 50 mm diameter. Thus, a factor of 0.93 was applied to the measured compressive strength to account for size effect (Mindess, Young et al. 2003). Results from the material tests are shown in Table 3.7 and Table 3.8. The tensile strengths presented in Table 3.8 were the values directly obtained from splitting tensile tests. No factor was applied to the tensile strength as the cylinder diameter has little effect on tensile splitting strength of concrete (Mitchell 1961).

Fig. 3.7 Compressive and tensile splitting tests

Table 3.7 Compressive strengths of cylindrical cores

Slab ID.	Compressive cylinder tests			
	No. of cylinder specimens	Factored average strength (MPa)*	Standard deviation (MPa)	Elastic Modulus (GPa)
HC1.320	3	54	6	34.9
HC2.400	1	57	n.a	n.a
HC3.500	2	56	4	35.5
HC4.320	3	56	6	35.2

* A factor of 0.93 was applied

Table 3.8 Tensile strengths of cylindrical cores

Slab ID.	Tensile splitting tests		
	No. of cylinder specimens	Average strength (MPa)	Standard deviation (MPa)
HC1.320	2	3.6	0.36
HC2.400	2	4.2	0.45
HC3.500	2	4.1	0.14
HC4.320	3	4.0	0.23

Elastic modulus of concrete was calculated from stress-strain curves determined from the compressive cylinder tests and reported in Table 3.7. During the compressive strength

tests, strain gages affixed to concrete samples of HC2.400 did not function properly. Therefore, neither strain nor elastic modulus data were obtained for this slab. For this reason, the concrete elastic modulus of this slab was assumed to be 35.2 GPa, a mean value of the other three slabs.

3.2.3 Test rig and instrumentation

Since no loads were applied in the heat transfer test as mentioned in Section 3.2.1, test setup for the heat transfer test (H.HC.400) will be introduced separately in Chapter 5. This section shows an overall setup for the other eight structural tests in Series I and II. A 4-point loading scheme was employed, of which all the slabs were loaded by two line loads placed symmetrically with respect to the slab center (Fig. 3.8). In Series I, all the four specimens were subjected to an increasing load till failure. On the other hand, the four specimens in Series II were first loaded up to predetermined levels (25 or 40% of their ultimate capacity defined by tests at ambient condition in Series I (Table 3.2)). Heat was subsequently applied. The load levels remained unchanged during the heating time and the tests were terminated once failure occurred.

Fig. 3.8 Test rig of Series I & II

Instrumentation in Series I included load cells (LCs) and linear variable displacement transducers (LVDTs). Load cells and LVDTs were arranged to monitor transferred loads

from a hydraulic jack to the specimen and vertical displacement developments along the slab length, respectively. In Series II, besides LCs and LVDTs, type K thermocouples were used. These thermocouples were embedded to measure temperature evolution inside the slabs during the fire tests. In addition, crack propagation and failure mode were visually tracked during the tests.

3.3 Experimental program on FR PCHC slabs (Series III & IV)

3.3.1 An overview of testing program

The literature review in Chapter 2 (Section 2.4) shows that among shear-strengthening techniques, the use of fibers has attracted the most attention. However, it is shown that previous studies were limited to the use of hooked steel (H) fibers while other types of steel fiber have not been examined. Moreover, the upper limit of hooked steel fibers used in very dry concrete in the extrusion method has not been quantified. Furthermore, although the use of PP fibers to strengthen shear capacity of PCHC slabs exhibited compelling improvements, there has been a limited number of test results. Besides, the effectiveness of using fibers to enhance shear resistance under fire conditions has not been investigated yet.

To address these shortcomings, an experimental program on shear behavior of FRC hollow-core slabs at both room and elevated temperatures was carried out. Three types of fiber including PP, high-strength/straight (S) and hooked steel fibers (Fig. 3.9) with different contents were examined. At first, the use of high-strength/straight steel fibers to strengthen shear capacity and to overcome concrete compaction problems in the extrusion method caused by high contents of hooked steel fiber was investigated. In addition, it should be noted that for each type of specimen (six types in total), two identical slabs were cast, of which one was tested at room temperature (Series III) while the other was evaluated under fire conditions (Series IV). The investigations aimed to:

- (1) Experimentally quantify the effectiveness of incorporating PP, hooked and high-strength/straight steel fibers with different contents on shear strength of PCHC slabs at ambient and under fire conditions;
- (2) Investigate potential uses of such fibers in extruded hollow-core slabs in common practice.

Fig. 3.9 Fibers used in the experimental program

A summary of the test programs on FRC hollow-core slabs at ambient and under fire conditions is respectively shown in Table 3.9 and Table 3.10. It is worth mentioning that steel-fiber contents of 40 and 70 kg/m^3 corresponding to 0.51 and 0.89% by volume as shown in the two tables were adopted in this study to be in accordance with recommended amounts of steel fibers used in SFRC members from previous studies (Victor C. Li and Ali 1992, Peaston and Paine 1999, Yoon-Keun Kwak and Jubum 2002, Hai H. Dinh and James 2010, Palmer and Schultz 2011, Cuenca and Serna 2013, Simasathien and Chao 2015, Dudnik, Milliman et al. 2017). The amount of PP fiber, however, was selected based on a common practice in precast plants, of which 1 to 2 kg/m^3 of PP fibers (corresponding to 0.11 and 0.22% by volume) is usually added to improve the resistance of concrete to explosive spalling in the event of fire.

Table 3.9 Summary of test program on FRC hollow-core slabs at ambient (Series III)

Slab ID.	Test ID.	Fiber type	Fiber content in kg/m ³ (by volume, %)	Shear span (mm)	Test load
HC1.0	HC1.0.a	n.a	n.a	1000	To failure
	HC1.0.b	n.a	n.a	1000	To failure
HC2.PP.1	HC2.PP.1.a	PP fiber	1 (0.11)	1000	To failure
	HC2.PP.1.b	PP fiber	1 (0.11)	1000	To failure
HC3.PP.2	HC3.PP.2.a	PP fiber	2 (0.22)	1000	To failure
	HC3.PP.2.b	PP fiber	2 (0.22)	1000	To failure
HC4.H.40	HC4.H.40.a	H fiber	40 (0.51)	1000	To failure
	HC4.H.40.b	H fiber	40 (0.51)	1000	To failure
HC5.H.70	HC5.H.70.a	H fiber	70 (0.89)	1000	To failure
	HC5.H.70.b	H fiber	70 (0.89)	1000	To failure
HC6.S.70	HC6.S.70.a	S fiber	70 (0.89)	1000	To failure
	HC6.S.70.b	S fiber	70 (0.89)	1000	To failure

Table 3.10 Summary of test program on fire-exposed FR PCHC slabs (Series IV)

Slab ID.	Fiber type	Fiber content in kg/m ³ (by volume, %)	Shear span (mm)	Testing condition	Test load
F.HC1.0	n.a	n.a	1000	in fire	40% of HC1.0
F.HC2.PP.1	PP fiber	1 (0.11)	1000	in fire	40% of HC2.PP.1
F.HC3.PP.2	PP fiber	2 (0.22)	1000	in fire	40% of HC3.PP.2
F.HC4.H.40	H fiber	40 (0.51)	1000	in fire	40% of HC4.H.40
F.HC5.H.70	H fiber	70 (0.89)	1000	in fire	40% of HC5.H.70
F.HC6.S.70	S fiber	70 (0.89)	1000	in fire	40% of HC6.S.70

As shown in Table 3.9, except for HC1.0 (the control specimen), all the slabs in Series III were designated as HCX.Y.Z, where X denoted the order of casting the specimen, Y

referred to the type of fiber used (PP, H, or S), and Z indicated fiber content in kg/m^3 . For example, HC2.PP.1 referred to the second specimen to be cast in Series III. In addition, the specimen contains PP fibers with a content of 1 kg/m^3 . Similarly, the five specimens in Series IV were named as F.HCX.Y.Z. Here X, Y, Z had the same meanings with those in Series III while F was prefixed to indicate that the test was conducted under fire conditions (Table 3.10). In Series III, HC1.0 was the control specimen. The two slabs with PP fibers (HC2.PP.1 and HC3.PP.2) were cast to study shear behavior of PCHC slabs using PP fibers with two different PP fiber contents (1 kg/m^3 vs 2 kg/m^3). Similarly, the two slabs with hooked steel fibers (HC4.H.40 and HC5.H.70) were tested to investigate shear response of the slabs with steel fibers, and to compare the effectiveness of using different contents of hooked steel fibers (40 kg/m^3 vs 70 kg/m^3). Furthermore, the slab with high-strength/straight fibers (HC6.S.70) was to study behavior of PCHC slabs using straight fibers and to compare the effectiveness of hooked steel fibers (HC5.H.70) vs straight fibers (HC6.S.70) on shear resistance when the same fiber content (70 kg/m^3) was used. The six respective specimens in Series IV (compared to Series III) were conducted for the same purposes. However, while Series III was to investigate shear behavior at ambient, Series IV was to study shear response under fire conditions. In general, the research compared the use of three types of fiber (hooked steel, high-strength/straight steel and pp fibers) with different contents so as to evaluate the potential of each type of fiber in terms of improving shear resistance of PCHC slabs.

To utilize FRC specimens and to maximize test data, both ends of the six slabs in Series III were tested using a 3-point loading scheme (Fig. 3.17(a)), resulting in two independent tests, namely, HCX.Y.Z.a and HCX.Y.Z.b as shown in Table 3.9, in which HCX.Y.Z.a was always conducted first. After HCX.Y.Z.a was carried out, Slab HCX.Y.Z was rotated by 180° about its vertical axis to bring the untested end to the other side where load was applied for the second test (HCX.Y.Z.b). Therefore, a total of twelve tests were carried out on six hollow-core specimens resulting in two sets of test data for each type of fibers used in Series III. However, such a practice could not be applied under fire conditions as the material properties of the specimen would have significantly altered

after being subjected to fire in the first test. Thus, only six fire tests were conducted in Series IV. Testing procedure and heating curves of Series IV followed the structural fire tests in Series II, of which the specimens were pre-loaded up to 40% of their ultimate capacity defined by ambient tests and heat was subsequently applied. The furnace and the fire curve shown in Fig. 3.2 were also adopted for the six fire tests in Series IV.

3.3.2 Test specimens of Series III and IV

3.3.2.1 Hollow-core specimens

Series III and IV employed section properties of the 400 mm deep slab in Series I & II. Therefore, all the specimens in Series III and IV were 4.15 m long, 1.2 m wide with depth of 400 mm. They comprised 4 non-circular hollow-cores as shown in Fig. 3.10 and were fabricated using the extrusion process.

Fig. 3.10 Nominal geometric properties of test slabs

3.3.2.2 Material properties

All the specimens in Series III & IV were prestressed with two types of strand (9.6 mm and 12.9 mm diameters) which were identical to those in Series I & II. Properties of these strands are already introduced in Section 3.2.2.2. Therefore, only concrete properties of FRC hollow-core specimens are discussed in this section.

Concrete mixes with fibers

Three types of fiber as shown in Fig. 3.9 were incorporated in the concrete mixes to produce the fiber-reinforced concrete hollow-core specimens. Table 3.11 gives mix

proportions used in the six specimens of Series III, and of Series IV as well. It is noteworthy that fresh concrete used in the extrusion method must be very dry (zero- or nearly-zero-slump concrete). This requirement posed challenges in adding fibers into fresh concrete PCHC slabs. The inclusion of fibers introduces new surface area to react with water into the fresh concrete and the amount of water should be increased to obtain similar workability. However, to achieve a target compressive strength, the water-to-cement ratio should remain unchanged. A number of trial mixes were proposed and operated at a precast-yard lab and the final mixes that achieved all requirements of concrete to be used in the extrusion process were adopted for casting the specimens. Fig. 3.11 typically shows fresh concrete of one specimen with steel fibers cast using the extrusion method whereas Fig. 3.12 shows hardened FRC hollow-core specimens ready for testing. It is shown that by changing the dosage of admixture, it was feasible to add fibers into nearly-zero-slump concrete in the extrusion process without compromising on the target concrete strength of 48 MPa (cylinder strength).

Table 3.11 Mix proportions of Series III & IV

	Cement (OPC) (kg/m ³)	Water (liter/m ³)	Sand (kg/m ³)	Crushed aggregate with a max size of 15 mm / 20 mm (kg/m ³)	Admixture I / II* (liter/m ³)	Fiber type/content (kg/m ³)
HC1.0	440	130	859	661 / 356	2.64 / 0.88	n.a / 0
HC2.PP.1	440	130	859	661 / 356	3.52 / 0.88	PP / 1
HC3.PP.2	440	130	859	661 / 356	4.40 / 0.88	PP / 2
HC4.H.40	440	130	859	661 / 356	3.52 / 0.66	Steel (H) / 40
HC5.H.70	440	130	859	661 / 356	4.84 / 0.66	Steel (H) / 70
HC6.S.70	440	130	859	661 / 356	4.84 / 0.66	Steel (S) / 40

* Admixture I = MGA 8537 type; Admixture II = Matrix 1100 type

Fig. 3.11 Fresh FRC hollow-core specimen cast using the extrusion method

Fig. 3.12 FRC hollow-core slabs for testing

Material tests

Three types of material test including flexural tensile strength (for steel-fiber-reinforced concrete), compressive and tensile splitting tests were conducted. The first type of material tests was carried out at the factory 2 months after the specimens were cast. It was performed on a notched beam with a size of $150 \text{ mm} \times 150 \text{ mm} \times 750 \text{ mm}$, following EN 14651 (EN-14651 2005). Fig. 3.13 shows a flexural tensile strength test; the test results of all three specimens using steel fibers (HC4.H.40, HC5.H.70, and HC6.S.70) are provided in Table 3.12.

a) Flexural tensile strength test (HC4.H.40)

b) Typical load F-CMOD curve for fiber reinforced concrete (adapted from EN-14651 (2005))

Fig. 3.13 Flexural tensile strength test following EN-14651 (2005)

Table 3.12 Flexural tensile strength of fiber-reinforced concrete according to EN-14651

	$f_{R,1}$ (MPa)	$f_{R,2}$ (MPa)	$f_{R,3}$ (MPa)	$f_{R,4}$ (MPa)
HC4.H.40	8.7	10.6	10.6	9.4
HC5.H.70	11.0	11.8	9.6	8.1
HC6.S.70	8.2	6.6	6.0	4.9

$f_{R,j}$: residual flexural tensile strength corresponding to different CMODs as shown in Fig. 3.12(b).

The concrete compressive strength for the slabs, on the other hand, was measured from two 150 mm cube samples (except for HC6.S.70) at about 10 months after casting (Fig. 3.14). The cube strength results were then converted to cylinder strength following Table 3.1 - Eurocode 2 (EN-1992-1-1 2004). The cube samples of HC6.S.70, however, were misplaced at the factory and were not available for material tests. Therefore, compressive strength tests for HC6.S.70 were conducted on three cylindrical cores of 50 mm × 100 mm extracted from the specimen after testing (at ambient temperature in Series III). Similarly, tensile splitting strength results of all the six specimens were defined by a mean value of three cylinders with dimensions of 50 mm x 100 mm, which were also obtained by the coring method (Fig. 3.15). The measured values of the compressive and the tensile splitting strengths are presented in Table 3.13.

Fig. 3.14 Concrete compressive strength measured on cube samples

Fig. 3.15 Tensile splitting tests on cylindrical core samples

Table 3.13 Concrete properties

Slab ID.	Fiber type	Fiber content (kg/m ³)	Mean compressive cylinder strength (SD*) (MPa)	Mean tensile splitting strength (SD*) (MPa)
HC1.0	-	-	66.8 (1.0)	4.5 (0.1)
HC2.PP.1	pp fiber	1	79.1 (5.4)	7.4 (1.6)
HC3.PP.2	pp fiber	2	79.3 (5.9)	8.0 (1.5)
HC4.H.40	hooked steel fiber	40	65.3 (3.5)	7.2 (0.9)
HC5.H.70	hooked steel fiber	70	61.2 (5.7)	12.7 (2.4)
HC6.S.70	straight steel fiber	70	62.4 (10.2)	12.6 (0.7)

* SD = Standard deviation

To examine the quality of hardened concrete containing different types of fiber, cylindrical cores extracted from the six hollow-core specimens after they were tested at ambient temperature (Series III) were compared (shown in Fig. 3.16). As can be seen, concrete of the two specimens with PP fibers (HC2.PP.1 and HC3.PP.2) was less porous compared to concrete of the control specimen (HC1.0). However, the porosity of concrete increased with the steel-fiber content. These observations agree with the results from the concrete compressive strength tests shown in Table 3.13. The table shows that compressive concrete strength increased with the use of PP fibers. However, it slightly decreased with increasing steel-fiber content (65.3 vs 61.2 MPa). In general, it is shown that PP fibers improved both the compressive and tensile properties of concrete. However, only the tensile behavior was significantly improved with the inclusion of steel fibers (Table 3.13).

Fig. 3.16 Cylindrical cores extracted from the specimens (Series III)

3.3.3 Test rig and instrumentation

To utilize FRC hollow-core specimens such that both ends of the six slabs in Series III could be tested as mentioned in Section 3.3.1, a 3-point loading scheme was adopted. The setup of Series III followed the shear-test arrangement for PCHC slabs specified in EN 1168 (EN-1168:2005+A3 2011). All the specimens were simply supported at their two ends and loaded by an unsymmetrical line load with respect to the slab center. Shear span was taken as $2.5h$ (in which h is the overall slab depth, equal to 400 mm in this case) as specified in the provision, resulting in a shear span of 1000 mm in all of the tests. The test setup of Series IV was similar with that of Series III except that the specimens

of Series IV were placed on top of the electric furnace and the tests were conducted under fire conditions. Fig. 3.17 shows the test rig of Series III & IV. Similar to Series I, all specimens in Series III were subjected to an increasing load up to failure. On the other hand, the six specimens in Series IV were first loaded up to 40% of their ambient capacity defined by the tests in Series III and then heated up to failure. The load level of 40% was maintained during the heating time. Instrumentation in Series III & IV was similar to those in Series I & II, respectively. Detailed test setup and instrumentation of the specimens in Series I, II, III, IV are provided in Chapters 4, 6, 7 and 8, respectively.

Fig. 3.17 Testing arrangement (Series III & IV)

3.4 Summary

In this chapter, an experimental program including 21 specimens divided into 4 series to study shear behavior of deep PCHC slabs with and without fibers at both ambient and elevated temperatures is presented. Geometric and material properties of the specimens are provided in detail. In addition, test setup, instrumentation, heating conditions and testing procedure are introduced. Detailed information comprising test setup, experimental results, observations and discussions of each test series is presented in Chapter 4 (Series I), Chapter 6 (Series II), Chapter 7 (Series III) and Chapter 8 (Series IV). Results from the experiments were used to (1) study shear mechanism of deep PCHC slabs; (2) to verify FE models developed to simulate shear response of PCHC slabs at ambient and under fire conditions; (3) to validate shear strength formulas in some design

codes such as ACI 318, Eurocode 2, EN 1168; and (4) to develop semi-empirical models to predict shear strength of PCHC slabs without any fibers and with steel fibers.

In the following chapter (Chapter 4), experimental studies on 4 deep PCHC slabs of Series I are discussed. Code comparisons and numerical investigations based on the test results are presented afterward. The chapter also includes parametric studies based on verified FE models to examine several factors which significantly affect web-shear resistance of PCHC slabs.

CHAPTER 4 EXPERIMENTAL AND NUMERICAL STUDIES ON WEB-SHEAR BEHAVIOR OF DEEP PCHC SLABS AT ROOM TEMPERATURE (SERIES I)

4.1 Introduction

Following the objectives, geometric and mechanical material properties introduced in Chapter 3 (Section 3.2), this chapter presents detailed shear test setup, experimental results of the four specimens tested to failure in Series I. Code comparisons based on data obtained from the tests are also discussed. Numerical studies are subsequently introduced, followed by parametric studies to extend the range of deep PCHC slabs subjected to shear. The material presented in this chapter can be find in the references (Nguyen, Tan et al. 2017, Nguyen, Tan et al. 2019). The main objectives of Series I are:

- (1) To investigate web-shear mechanism of deep PCHC slabs;
- (2) To use the test data to evaluate the validity of some popular design codes for web-shear strength of deep PCHC slabs;
- (3) To use the experimental results to verify an FE model developed to simulate web-shear response of PCHC slabs. The model, in turn, can be used to further investigate web-shear resistance of PCHC slabs and to explain why deep PCHC slabs are more susceptible to web-shear failure.

In addition, the test data from Series I serves as a basis for the fire shear tests on deep PCHC slabs under fire conditions which are introduced in Chapter 6.

4.2 Experimental investigation

4.2.1 Test setup

Fig. 2.1 shows the overall shear-test setup for the PCHC slabs in the experimental program. All the specimens were simply supported at their two ends by a bearing system that included rubber pads, steel rollers, and concrete blocks. The hollow-core slabs were loaded by two line loads placed symmetrically in reference to the center line of the specimens. Rubber pads with a thickness of 10 mm and a width of 100 mm were placed between the hollow-core units and the steel rollers, as well as between the bearing supports and the specimens, to compensate for unevenness of the hollow-core surface, and to allow uniform stress distribution at these contact regions. Shear span varied from 600 to 800 mm to ensure that all hollow-core slabs would fail in shear instead of flexure. The superimposed load was applied using a hydraulic jack with a maximum capacity of 200 ton. Actions transferred to the loading beam and to the specimens were monitored by three load cells (LC1 to LC3). Vertical deflections were measured along the specimen length using five linear variable displacement transducers (LVDTs) (Fig. 2.1). The PCHC slabs were loaded to failure. Crack propagation and failure mode were also monitored during the test.

Fig. 4.1 Testing arrangement

4.2.2 Test results

Test results

Fig. 4.2 shows the load-displacement curves determined from the readings of load cells and LVDTs, and Table 2.1 summarizes the test results. It should be noted that the mid-span displacements shown in Fig. 4.2 and Table 2.1 are the relative deflections between the middle slab (LVDT 1) and the support locations (average values of LVDT 4 and LVDT 5). As can be noticed from Fig. 4.2, all the specimens responded to the imposed loads in elastic manner up to failure. Maximum displacements at failure were especially small, at less than or around 2 mm (Table 2.1). As there were no stirrups in the slabs, cracking loads were identical to failure loads, and the slabs failed suddenly without any warning.

Fig. 4.2 Load-displacement curves

Table 4.1 Test results

	Shear span (mm)	a/d	Failure load (kN)	Mid-span deflection at failure (mm)	Failure mode
HC1.320	800	2.86	184	0.9	Flexural-shear
HC2.400	800	2.25	511	1.3	Web-shear
HC3.500	800	1.79	1162	2.1	Web-shear
HC4.320	600	2.25	194	1.1	Flexural-shear

Failure modes

As the shear span varied from 600 to 800 mm, two different modes of failure were perceived in the four tests. HC1.320 and HC4.320 failed due to a combination of shear and flexure. Flexure cracks started off and then propagated diagonally upwards toward the applied-load location, resulting in a damage of concrete in the compression zone, illustrating flexural-shear failure. On the other hand, HC2.400 and HC3.500 collapsed as the maximum principal stress (tension) in the concrete webs close to the support exceeded tensile strength of concrete. A diagonal crack occurred and immediately propagated in both directions toward and the bearing region and the applied-load location, causing a sudden failure, demonstrating web-shear failure (Nguyen, Tan et al. 2017). Fig. 4.3 depicts the two failure modes observed in the four tests.

Fig. 4.3 Failure modes in PCHC slabs

4.3 Shear strength of PCHC slabs

4.3.1 Bond stress and transmission length of prestressing strands

An accurate estimation of transmission length (or transfer length) is critical for shear strength predictions of PCHC slabs as it directly affects the amount of prestressing force

applied to the concrete at considered sections. In this study, transmission lengths were computed using ACI 318-14 (ACI 2014), Eurocode 2 (EN-1992-1-1 2004) and EN 1168 (EN-1168:2005+A3 2011), as shown in Table 4.2. These values will be used in the theoretical predictions by the corresponding codes in the next section. It is noteworthy that calculations for transmission length in EN 1168 follow the code provisions in Eurocode 2 as EN 1168 is a harmonized European Standard, aiming for hollow core slabs. It is shown that compared to Eurocode 2 and EN 1168, ACI 318 requires a greater transmission length for the strands to develop their effective stress.

Table 4.2 Transmission length

Slab ID.	Strand type	Transmission length (mm)		
		ACI 318-14	Eurocode 2	EN 1168
HC1.320	9.6 mm strand	475	313	313
HC2.400	9.6 mm/12.9 mm strand	475/638	302/406	302/406
HC3.500	12.9 mm strand	638	410	410
HC4.320	9.6 mm strand	638	313	313

4.3.2 Shear capacity of hollow core specimens

The shear capacity of the PCHC specimens was calculated using Eurocode 2 (EN 1992-1-1: 2004 Section 6.2.2), EN 1168:2005+A3:2011 (Sec. 4.3.3.2.2), and ACI 318-14 (Sec. 22.5.8.3) and is shown in Table 4.3. Such results are put in comparisons with the experimental data and presented in Table 4.4. It should be noted that in these calculations and comparisons, the mean strengths of concrete as shown in Chapter 3 (Tables 3.7 and 3.8) instead of characteristic strengths were used. In addition, safety factors including material partial safety factors and strength reduction factors were set to unity. Moreover, the comparisons between the experimental data and calculations using the design codes are based on test result of one single specimen for each case, therefore, results from these comparisons are non-statistical.

The comparisons from Table 4.4 indicated that ACI 318-14 overpredicts the shear strength of the two specimens failed in flexural-shear. While a factor of 0.5 was applied to the web-shear capacity, the code provides conservative predictions for the two slabs that failed in web-shear, i.e., HC2.400 and HC3.500. However, the failure load of HC3.500 was considerably higher than the web-shear strength computed using ACI 318-14. This fact is probably owing to the effect of a/d . For short a/d , shear is transferred by arching action in lieu of beam action as shown in previous studies (Tan, Kong et al. 1995, Wright and MacGregor 2009). In the experimental program, the a/d values of HC1.320, HC2.400, and HC4.320 were 2.86, 2.25 and 2.25, respectively. At such ratios, arching action had less effect on shear resistance. Nonetheless, with a/d of 1.79, HC3.500 was significantly affected by the arching action, producing a much higher shear-carrying capacity compared to the code prediction (Nguyen, Tan et al. 2019). Contrary to ACI 318-14, Eurocode 2 and EN 1168 provide safe predictions for the two slabs failed in flexural-shear. However, Eurocode 2 overpredicts two specimens that failed in web-shear, while EN 1168 overpredicts the web-shear capacity of HC2.400.

Table 4.3 Shear strength computed by design codes

Slab ID.	Exp. Shear force (kN)*	Exp. Failure mode	Predicted shear capacity (kN)		
			ACI 318 - 14	Eurocode 2	EN 1168
HC1.320	204.7	Flexural-shear	352.7	114.5	114.5
HC2.400	533.2	Web-shear	292.2	1009.4	790.1
HC3.500	1186.7	Web-shear	403.5	1352.8	1055.4
HC4.320	216.5	Flexural-shear	470.8	115.7	115.7

* Including slab weight and the loading system

Table 4.4 Shear-strength comparisons: Experimental results vs. code formulas

Slab ID.	a/d	Exp. Shear force (kN)	$V_{test}/V_{calculation}$		
			ACI 318 - 14	Eurocode 2	EN 1168
HC1.320	2.86	204.7	0.58	1.79	1.79
HC2.400	2.25	533.2	1.82	0.53	0.67
HC3.500	1.79	1186.7	2.94	0.88	1.12
HC4.320	2.25	216.5	0.46	1.87	1.87

4.4 Numerical studies

4.4.1 FE modeling for web-shear behavior of PCHC slabs

4.4.1.1 Modeling technique

To investigate web-shear behavior of PCHC slabs, three-dimensional (3D) FE models were created using ABAQUS/Explicit (2014) (Simulia 2014). Concrete was modeled employing 8-node linear brick elements (C3D8R). Material and section properties of HC2.400 were adopted to build the FE model. Fig. 4.4 shows the concrete element mesh of HC2.400 in vertical and longitudinal directions.

Fig. 4.4 Element mesh of HC2.400: a) in vertical direction; b) in longitudinal direction

Prestressing steel was explicitly modeled by truss elements (T3D2). A perfect bonding condition between concrete and prestressing steel was assumed (Fig. 4.5). Strand stress was postulated to be zero at the free ends of the slab and to achieve the full effective stress at the end of transmission length. Despite the fact that bond stress (proportional to

the rate of change of strand stress over transmission length) is not uniform but decreases nonlinearly from the free end within the transmission length (Uijl 1998, Ramirez-Garcia, Dang et al. 2017), a linear distribution in strand stress was assumed in this study. Eurocode transmission length as shown in Table 4.2 was adopted in this modeling.

Fig. 4.5 FE modeling of PCHC slabs

4.4.1.2 Material model for concrete

ABAQUS (2014) has proposed three concrete models, i.e. *concrete smeared cracking*, *cracking model for concrete*, and *concrete damage plasticity*. Among the three, *concrete smeared cracking* model is commonly used for concrete subjected to essentially monotonic straining at low confining pressures (Simulia 2014). The model can be used in ABAQUS/Standard package. The *cracking model for concrete*, on the other hand, is developed for brittle cracking modeling with the assumption that compressive behavior is constantly linear elastic. The model can be employed for simulating other materials such as ceramics or rock in which brittle failure dominates the behavior (Simulia 2014). This model is available in both ABAQUS/Standard and ABAQUS/Explicit. The third model, *concrete damage plasticity*, offers a general capability for simulating concrete and other quasi-brittle materials. In the *concrete damage plasticity* model, two failure mechanisms are specified: concrete will fail either under compression crushing or tension

cracking (Simulia 2014). The model can be applied in both ABAQUS/Standard and ABAQUS/Explicit options.

In this study, *Concrete damage plasticity* model (CDP model) was selected for concrete material as it can describe the concrete nonlinear behavior more appropriately, considering the interface behavior between the strands and the concrete. It should be noted that the CDP model, developed from classical plasticity theory, includes three components, i.e., a yield criterion that describes the initiation of yielding, a flow rule that relates the stress increments to the plastic strain increments after yielding, and a hardening (softening) rule that predicts changes in the yield surface. However, in the CDP model, the hardening (softening) variable is displaced by a plastic-damage variable, k , that never decreases but only increases from zero to unity once plastic deformation takes place (Lubliner 2013).

As mentioned in Nguyen, Tan et al. (2017), the constitutive compressive concrete model in uniaxial stress state consists of ascending and descending portions. The *fib* model code (FIB 2010) was adopted to simulate the concrete compressive behavior prior to the peak stress. The post-peak portion, which is highly sensitive to the specimen dimensions and testing conditions (Lubliner 2013), was obtained by adopting Van Vliet and Van Mier's model (Van Vliet and Van Mier 1995). In the model, the effect of specimen dimension on softening behavior of concrete in compression was taken into account. Tensile behavior of concrete was described as stress-crack opening relationship to overcome difficulties in convergence caused by mesh sensitivity (Demir, Öztürk et al. 2016). The ascending part of the constitutive model in tension was presumed to be linear. In addition, stress-crack opening function proposed by Hordijk (1992) was employed to simulate the softening behavior of concrete in tension. In the CDP model, only stress-inelastic strain/crack opening relation part is required to be input (Simulia 2014). Fig. 4.6(a) and Fig. 4.6(b) respectively show behavior of concrete in compression and tension for HC2.400 adopted in the FE model.

Fig. 4.6 Constitutive model for concrete

The curves presented in Fig. 4.6 describe the response of concrete under uniaxial stress state. In the CDP model, behavior of concrete in bi-axial and tri-axial stress states are defined by four plastic parameters, viz. ratio of initial biaxial compressive yield stress to initial uniaxial compressive yield stress (f_{b0}/f_{c0}), ratio of the second stress invariant on tensile meridian to that on the compressive meridian (K_c), eccentricity (ϵ), and dilation angle (ψ). It should be noted that as Abaqus/Explicit was employed, viscosity, the fifth (also the last parameter) in the CDP model, was ignored and taken as 0 as recommended by Abaqus. Physical meanings of the first four plastic parameters and sensitivity of numerical predictions to these parameters will be discussed in the next section.

4.4.2 Model sensitivity study

Sensitivity of numerical analyses was conducted on different mesh sizes of concrete and different values of the four plastic parameters in the CDP model. Material and geometric properties of HC2.400 were employed to conduct the model sensitivity study.

4.4.2.1 Mesh convergence study

This section presents an investigation of the effect of mesh size on FE model prediction accuracy. Technically, concrete mesh size should not be smaller than the strand diameter. In addition, the mesh size should not be larger than the web thickness. Therefore, three

different mesh sizes of 20, 30, and 40 mm in the region between the slab's free edge and the applied-load locations were used. Only one coarser mesh size of 40 mm was used in the middle slab region since it is less critical in the web-shear behavior. Fig. 4.7 shows the three different mesh sizes investigated in the numerical analyses. Fig. 4.8 plots a comparison of load-displacement relationships for the three different mesh sizes. It shows that the FE models employing these three meshes reasonably capture the shear behavior of HC2.400 in terms of brittle failure and peak load. The differences in the initial stiffness between the numerical and experimental results will be discussed in detail in Section 4.4.3. Suffice to say, the deflection at failure for HC2.400 was only 1.3 mm, but the initial slack due to gaps in the contact regions was 0.5 mm as shown in Fig. 4.8.

Fig. 4.7 Different mesh sizes used for structural analysis: a) 20x40mm; b) 30x40mm; c) 40x40mm

Fig. 4.8 Mesh-size comparisons for HC2.400

As can be seen in Fig. 4.8, failure loads from the FE models using the mesh sizes of 20 mm and 30 mm were almost identical (498 kN vs 499 kN). The predicted values were close to the failure load of 511 kN with an error of 2%. The predicted shear strength from the FE model using the mesh size of 40 mm was 533 kN, only 4% larger than the test result. Therefore, it can be concluded that all the three mesh sizes of 20, 30, and 40 mm are suitable for modelling the shear behavior of 400 mm thick hollow-core slabs. At the end, the mesh size of 40 mm was selected because the model required less computational time and had satisfactory accuracy.

4.4.2.2 Yield function: f_{b0}/f_{c0} and K_c studies

The yield function in Abaqus CDP model has the form of:

$$F = \frac{1}{1 - \alpha} \left(\bar{q} - 3\alpha\bar{p} + \beta(\bar{\epsilon}^{pl})(\hat{\sigma}_{max}) - \gamma(-\hat{\sigma}_{max}) \right) - \bar{\sigma}_c(\bar{\epsilon}_c^{pl}) = 0 \quad (4.1)$$

where $\hat{\sigma}_{max}$ is the maximum principal effective stress; and $\bar{\sigma}_c(\bar{\epsilon}_c^{pl})$ is the effective compressive cohesion stress, accounting for degradation in concrete stiffness after plastic strain develops; α , β , and γ are dimensionless parameters in which α and β are functions of f_{b0}/f_{c0} defining the yield surface in biaxial stress state. Experimental data from Kupfer, Hilsdorf et al. (1969) showed that for normal-strength concrete, f_{b0}/f_{c0} lies between 1.10 and 1.16, resulting in α ranging from 0.08 to 0.12 (Fig. 4.9(a)). On the other hand, γ appears only in triaxial compressive state ($\sigma_{max} < 0$) and is a function of K_c , the ratio of the second stress invariant on the tensile meridian to that on the compressive meridian. The term K_c , which is in the range of 0.5 to 1, governs the shape and size of the yield surface in deviatoric plane (Fig. 4.9(b)). $K_c \geq 0.5$ to ensure that the yield surface is convex. When $K_c = 1$, failure surface in the deviatoric plan is a circle, representing the Drucker-Prager boundary surface.

Fig. 4.9 Yield surface (adapted from Abaqus (Simulia 2014) and Lubliner, Oliver et al. (1989))

Fig. 4.11 shows the plots of load-displacement curves from the FE models with different values of f_{b0}/f_{c0} and K_c . As expected, web-shear behavior was not sensitive to f_{b0}/f_{c0} and K_c as the stress state in concrete webs of hollow-core slabs is biaxial principal compression-tension as shown in Fig. 4.10, while f_{b0}/f_{c0} and K_c are more critical for concrete in biaxial and triaxial compressive states, respectively. That explains why even when f_{b0}/f_{c0} was increased up to 1.3 (unusual in normal-strength concrete), shear strength of HC2.400 remained constant (Fig. 4.11(a)). Therefore, for shear predictions of PCHC slabs, the Abaqus default values of 1.16 for f_{b0}/f_{c0} and 2/3 for K_c are adopted for the numerical studies.

Fig. 4.10 Stress state in concrete web

Fig. 4.11 Model sensitivity against yield function

4.4.2.3 Plastic flow: Eccentricity and dilation angle studies

The flow potential (G) in the CDP model follows the Drucker-Prager hyperbolic function as follows:

$$G = \sqrt{(\epsilon \sigma_{t0} \tan \psi)^2 + \bar{q}^2} - \bar{p} \tan \psi \quad (4.2)$$

where σ_{t0} is the uniaxial tensile stress at failure; ϵ is an eccentricity; and ψ is the dilation angle defined in the hydrostatic-deviatoric (p - q) plane as illustrated in Fig. 4.12.

Fig. 4.12 Hyperbolic surface of the flow potential function in meridional plane

Eccentricity study

As shown in Fig. 4.12, eccentricity (ϵ) is the small distance between the vertex of the hyperbola and the intersection of its asymptotes. When ϵ goes to zero, the original Ducker-Prager hypothesis that the flow potential surface in meridional plane is a straight line is valid. In the CDP model, Abaqus recommends ϵ to be 0.1 to avoid convergence

difficulty when concrete is subjected to low confining pressure. Fig. 4.13 presents a comparison of load-deflection curves from the FE models with $\epsilon = 0$ and 0.1. The figure shows that shear behavior of PCHC slabs was not sensitive to eccentricity as the two curves were almost identical regardless of variations in this term. Therefore, the value of $\epsilon = 0.1$ was selected.

Fig. 4.13 Eccentricity sensitivity

Dilation angle study

As the Drucker-Prager hyperbolic function is adopted to represent the flow potential function, ψ in the CDP model can be interpreted as internal concrete friction angle (ϕ), i.e., the angle between the hydrostatic axis and the yield surface, measured in the meridional plane (Fig. 4.12). However, defining a suitable value for dilation angle in the CDP model is one of the most challenging tasks as Abaqus does not suggest a default value.

It is known that the usual dilatancy angle for plain concrete is 15° (Oñate, Oller et al. 1988). As reinforcement is incorporated, reinforced concrete (RC) material can carry greater loads, which means the yield surface can dilate more compared to plain concrete. Thus, the dilation angle in RC materials can be greater than 15° . Lubliner, Oliver et al. (1989) assumed $\phi = \psi = 15^\circ$ for plain concrete subjected to biaxial compression while ϕ

$\psi = 32^\circ$ was applied for notched beam problems. Similarly, Oñate, Oller et al. (1988) used $\phi = \psi = 32^\circ$ to analyze the behavior of prestressed cantilever beams subjected to concentrated loads at their free ends. Furthermore, $\psi = 56^\circ$ was used in problems of confined circular concrete columns and deep beams (Jiang and Wu 2012, Demir, Ozturk et al. 2016). Numerical results correlated well with experimental data in all these studies. Clearly, the values of ϕ and ψ vary from problems to problems.

Sümer and Aktaş (2015) used solid elements and the CDP model to capture flexural behavior of RC beams. The model sensitivity of such RC structures was checked against changes in dilation angle, showing that the flexural behavior was not dilation-angle sensitive. In spite of dilation-angle variation from 30° to 50° , numerical predictions almost remained unchanged and correlated well with test data. Thus, it can be inferred that dilation-angle is not a sensitive parameter in flexural problems. However, it is believed that web-shear behavior is highly sensitive to dilation-angle variations because shear resistance of PCHC slabs relies on strength of concrete, while different dilation angles will result in different maximum stresses of the flow potential surface in the p - q plane as shown in Fig. 4.14.

To check the sensitivity of shear capacity against variations of dilation angle, ψ was randomly chosen as 20° , 25° , 28° , 32° , and 40° . Fig. 4.14 shows a comparison of load-deflection curves of the FE models with different assumed values of ψ . As expected, the web-shear behavior of HC2.400 was highly sensitive to changes of dilation angle. Predicted failure load was dramatically increased from 312 kN to 631 kN as ψ increased from 20° to 40° (Fig. 4.14). The best-fit result was obtained with dilation angle of 28° for HC2.400 (498 kN in the FE model vs. 511 kN in the test). The study confirmed that web-shear behavior of PCHC slabs was sensitive to dilation angle, since the latter governs the maximum stress of the flow potential surface with different concrete confining pressures in the meridional plane. In addition to the study of Sümer and Aktaş (2015) for flexural behavior, it is believed that plastic dilation angle in the CDP model is not only a material

property, but also controls structural behavior. Certain problems could be highly sensitive to dilation angle variations, while others are not. The selection of ψ in FE models, therefore, must be validated by actual tests to provide sensible predictions.

Fig. 4.14 Dilation angle sensitivity

4.4.3 Model validation

Since the FE models were developed to simulate the web-shear behavior of PCHC slabs, the two specimens which failed in web-shear, i.e., HC2.400 and HC3.500, were employed for model validations. Besides, the FE models were verified by 5 shear tests from Rahman, Baluch et al. (2012) and another 3 shear tests from Pajari (2005).

4.4.3.1 Specimens in the experimental program

Fig. 4.15 presents a comparison of the imposed load vs. mid-span deflection between the tests and the FE models, whereas Fig. 4.16 compares the failure mode and concrete damage at ultimate state of HC2.400 and HC3.500 from the experiments and the numerical models. It is noteworthy that concrete damages in the FE models were presented in term of concrete maximum plastic strain in tension. It can be observed from Fig. 4.15 that the load-versus-deflection curves and the brittle failures of the two slabs were adequately simulated in the FE models, except that the FE curves were stiffer than those obtained from the tests, showing a common challenge in shear modelling of

concrete structures (Demir, Ozturk et al. 2016). The lower stiffness value in the test was attributable to a slack in the test setup for the PCHC slabs, due to inevitable gaps at the contact regions. The discrepancy is reduced if the FE curve is shifted along the horizontal axis by the initial slack of 0.5 mm for HC2.400 and 0.4 mm for HC3.500 as shown in Fig. 4.15(a) and Fig. 4.15(b), respectively. However, most importantly, failure loads predicted by the FE models were in reasonable agreement compared to the test results, i.e., 2.5% for HC2. 400, and 13.5% for HC3.500. In addition, the web-shear failure was nicely captured in the FE models for both specimens (Fig. 4.16), showing the validity of the FE models for simulating web-shear behavior.

Fig. 4.15 Load-deflection comparison between tests and FE models

Fig. 4.16 Web-shear failure in the tests and FE models

4.4.3.2 Shear tests from Rahman, Baluch et al. (2012) and Pajari (2005)

The developed FE models were further validated by five shear tests on PCHC slabs from Rahman, Baluch et al. (2012). The PCHC units were 250 mm and 300 mm in depth, characterized by circular voids. 28-day concrete compressive strength was 40 MPa (cylinder strength). Besides, the FE models were also verified by another three shear tests on PCHC slabs namely, 133.320, 161.370, and 520.400 from Pajari (2005). In such notations, 320, 370, and 400 represented the slab thickness. All the eight slabs employed in this numerical model validation failed in the web-shear mode.

Generally, load-displacement curves were not shown in previous studies on shear behavior of PCHC slabs because specimens usually failed at very small displacements. In addition, the determination of shear capacity of PCHC specimens was more important because they behaved almost elastically. This was also the case for the five tests in Rahman, Baluch et al. (2012) and for the three tests in Pajari (2005). As load-displacement curves were not available, the validation was conducted by comparing the failure modes and failure loads between the tests and the FE models. Fig. 4.17 and Fig. 4.18 typically shows the model validation in term of failure mode using two of the five slabs from Rahman, Baluch et al. (2012). While Fig. 4.17 shows web-shear failure predicted by the FE models, Fig. 4.18 shows a comparison of failure modes for the 250 mm and 300 mm thick PCHC slabs from both experimental and numerical studies. The two figures clearly illustrate that the developed FE models were capable of simulating web-shear failure observed in the tests.

Fig. 4.17 Web-shear failure captured in the FE model (S13-2.5-300-1)

a) S13-2.5-300-1

b) S11-2.5-250-1

Fig. 4.18 Comparison of failure modes between experimental and numerical studies

Table 4.5 compares the failure loads obtained in the tests and those captured from the FE models using a dilation angle of 28° for all the specimens. It shows that the FE models resulted in conservative predictions for shear strengths of 7 out of the 10 PCHC slabs. In addition, the average discrepancy in failure loads between the actual tests and the FE models was in a satisfactory range of 3% with a CoV of 8.1%. In summary, the validation for a total of 10 slabs in three different test series demonstrated that the FE models could simulate web-shear response of PCHC slabs with different slab depths, web thicknesses and core configurations.

Table 4.5 Comparison of failure loads between experiments and FE models

	Slab I.D	a/d	V_{exp} (kN)	V_{pre} (kN)	V_{exp}/V_{pre}
From the experimental study	HC2.400	2.25	511	498	1.03
	HC3.500	1.79	1162	1007	1.15
From Rahman, Baluch et al. (2012)	S11-2.5-250-1	2.5	433	429	1.01
	S12-2.5-250-2	2.5	397	429	0.93
	S13-2.5-300-1	2.5	529	507	1.04
	S14-2.5-300-2	2.5	598	507	1.18
	S15-2.5-300-3	2.5	508	507	1.00
From Pajari (2005)	133.320	2.5	593	610	0.97
	161.370	3.1	564	608	0.93
	520.400*	2.4	345	328	1.05
Mean / CoV (%)					1.03 / 8.10

* 520.400 was loaded using 3-point loading scheme (unsymmetrically) instead of 4-point loading scheme (symmetrically) as in the rest of the specimens.

4.4.4 Influence of design variables – Parametric study

The verified FE model was employed to examine the effect of several design variables on web-shear resistance. Such variables included concrete strength, slab geometry, and level of prestressing. Material and geometric properties of HC2.400 were used to conduct parametric studies.

Strength of concrete

To examine the effect of concrete strength on the web-shear resistance of PCHC slabs, three FE models were developed with 3 concrete cylinder strengths of 40, 55, and 70

MPa. Concrete tensile strengths corresponding to such compressive strengths were calculated using Model Code 2010 (FIB 2010) as follows:

$$f_{ctm} = 0.3(f_{ck})^{\frac{2}{3}} \quad \text{for concrete grades} \leq \text{C50} \quad (4.3)$$

$$f_{ctm} = 2.12 \ln(1 + 0.1(f_{ck} + 8)) \quad \text{For concrete grades} > \text{C50} \quad (4.4)$$

Table 4.6 presents the values of concrete tensile strength and elastic modulus computed from Model Code 2010. Load-deflection curves obtained from the three numerical models are compared in Fig. 4.19, showing that web-shear failure occurred in all three models regardless of the variation in concrete strength. However, failure loads significantly increased with increasing concrete strength. The improvement in the web-shear capacity was due to an increase of compressive strength and tensile strength of concrete. Between the two, concrete tensile strength is more important as web-shear failure is assumed to occur when the maximum principal stress (tension) in the concrete web reaches the tensile strength of concrete. However, it was difficult to independently quantify the exact contribution of each characteristic because they were essentially linked to each other. Fig. 4.19 shows that the web-shear capacity of HC2.400 increased by 140% when concrete compressive strength increased from 40 MPa to 70 MPa and tensile strength increased by 31%. Results from the numerical investigations confirmed the vital role of concrete strength on web-shear resistance of PCHC slabs.

Table 4.6 Concrete properties

Slab ID.	Concrete compressive strength (MPa)	Tensile strength (MPa)	Elastic modulus (GPa)
Slab 1	40	3.51	36.3
Slab 2	55	4.21	39.7
Slab 3	70	4.61	42.6

Fig. 4.19 Load-displacement comparison

Slab geometry

Brunesi and Nascimbene (2015) showed that the mode of web-shear failure and shear stress distribution in PCHC slabs were highly sensitive to slab configurations such as hollow-core shape, web thickness, and concrete chords above and below the longitudinal voids. Therefore, to quantify the web-shear capacity with different core configurations, three FE models with different hollow-core geometries were developed. Fig. 4.20 shows detailed geometries of the three investigated slabs.

Fig. 4.20 Slab geometries: a) HC2.400; b) HC2.400a; c) HC2.400b

For the configurations studied here, results from the FE models indicated that the choice of angular or smoother-surface voids (Fig. 4.20) did not lead to a noticeable difference

in the web-shear strength. The maximum shear load resisted by HC2.400 (with angular cores) was 498 kN. Such value was only 1% larger than the shear capacity of HC2.400a with non-circular, smoother-surface voids (493 kN). However, more appreciable increase in the web-shear capacity was observed when comparing failure loads of HC2.400 and HC2.400b. As the web thickness increased from 55 mm in HC2.400 to 60 mm in HC2.400b (by 9%), the shear capacity increased by 7.2% (498 kN in HC2.400 vs. 535 kN in HC2.400b). It shows the vital influence of web thickness on the web-shear resistance.

Prestressing level

As it is widely accepted in traditional design approaches (ACI 318 or Eurocode 2), a higher level of prestressing force results in a higher shear capacity. However, Yang (1994) indicated that it was not necessarily true. Therefore, this section is to examine the influence of prestressing level on the web-shear resistance of PCHC slabs. Nine FE models were developed for HC2.400 with nine different levels of prestressing, ranging from 0% to 80%. The level of prestressing denoted the percentage of initial prestress applied to strands compared to their ultimate capacity. Fig. 4.21 shows the shear capacity of HC2.400 with different prestressing levels.

Fig. 4.21 Shear capacity vs. level of prestressing applied in strands

As shown in Fig. 4.21, with a lower level of prestressing force applied to strands (0 to 30%), flexural cracks occurred. The slab failed due to a combination of flexure and shear, resulting in flexural-shear failure. However, with a higher level of prestressing force (30% to 80%), web-shear failure occurred instead. These numerical results were consistent with previous studies (Sozen 1957, Pajari 2005) on web-shear mechanism of prestressed concrete members without shear reinforcement. Sozen (1957) observed that web-shear failure dominated in I-beams with thin webs and high prestressing force. Similarly, Pajari (2005) indicated that web-shear failure occurred in hollow core slabs which were heavily prestressed and subjected to a considerable shear force and limited bending moment.

In addition, Fig. 4.21 shows that a higher prestressing level benefited flexural-shear behavior as flexural-shear capacity increased with the level of prestressing. However, it brought unfavorable effect on web-shear resistance of PCHC slabs as web-shear strength decreased as the level of prestressing force increased. One explanation for such degradation in web-shear capacity of PCHC slabs is that, the critical point (where web-shear crack initiates) generally occurred within the transmission length region (shown in Fig. 4.22). As transmission length of strands was taken into account in the numerical study, there were additional horizontal shear stresses acting in the concrete web due to changes in the strand stress in the transmission length region as shown in Fig. 4.23. A higher level of prestressing force resulted in higher additional shear stresses in the concrete webs (Yang 1994). Therefore, the action effect on PCHC slabs increased. In addition, the stress state in concrete webs of hollow-core slabs is biaxial principal compression-tension (Fig. 4.10). A higher level of prestressing force results in a higher diagonal compression force in the concrete web. Since biaxial failure criterion was adopted in ABAQUS (Fig. 4.9(a)), for concrete under biaxial compression-tension stress state, allowable tensile stress to reach the yield surface decreases as compressive stress increases. Consequently, the tensile resistance of concrete webs decreased as the degree of prestressing force increased. Combining the action and resistance effects, web-shear strength of PCHC slabs decreased as the level of prestressing force increased.

The outcome of the numerical analyses on the web-shear resistance of PCHC slabs is also supported by experimental data of 7a and 10a on prestressed hollow core slabs tested by Walraven and Merx (1983). The two tests were identical, including the test setup, shear span, a/d , specimen geometry, and concrete strength, but they were different in terms of quantities of strands used. This difference in the number of strands led to different degrees of compression force transferred to the concrete strut. The experimental studies showed that the web-shear strength in test 10a which included 10 numbers of $\frac{1}{2}$ " strands were lower compared to that in test 7a which only consisted of 6 numbers of $\frac{1}{2}$ " strands (208.5 kN vs. 216.0 kN). A similar trend was also observed if tests 7b and 10b in the same study were compared.

4.5 Conclusions

An experimental investigation on shear response of deep precast, prestressed concrete hollow core (PCHC) slabs was conducted. Finite element analyses were performed to extend the understanding of web-shear mechanism of deep PCHC slabs. Based on these analyses, the following findings are drawn:

1. In some instances, deep PCHC slabs as the ones tested in this experimental program failed at loads lower than those predicted by EN 1168:2005+A3:2011, Eurocode 2, and ACI 318-14.
2. The Concrete Damage Plasticity model (CDP model) and the numerical techniques adopted in this study could be used to predict web-shear behavior of PCHC slabs with different cross-sectional geometries.
3. As dilation angle governed the maximum stress of the flow potential surface in the meridional plane, predicted shear capacity of HC2.400 increased by as much as twice when dilation angle increased from 20° to 40° . It showed that web-shear resistance was highly sensitive to dilation angle. Predicted failure load increased as dilation angle increased. Therefore, for numerical studies, the selection of dilation angle in FE models must be calibrated by actual test results before conducting any further numerical investigations.
4. Concrete strength played a dominant role in the web-shear resistance of PCHC slabs. From numerical studies, web-shear capacity increased by as much as 2.4 times as concrete strength increased from 40 to 70 MPa and tensile strength increased from 3.51 to 4.61 MPa.
5. In the simulated models, the change of hollow-core shape, from angular to smoother hollow core geometries, did not lead to a remarkable difference in web-shear capacity. However, the web-shear capacity significantly increased as web thickness was increased.
6. As transmission length of strands was taken into account in the numerical study, there were additional shear stresses acting in the concrete web due to changes in

the strand stress in the transmission length region. A high level of prestressing force induced higher shear stresses in the concrete webs. Besides, it also increased the magnitude of inclined principal compressive stress. The presence of a higher compression force reduced the tensile resistance of concrete webs in the direction of maximum principal tensile stress. As a result, web-shear strength decreased as the level of prestressing force increased.

7. The threshold at which prestressing force starts bringing negative effect on web-shear resistance of PCHC slabs depends on a number of variables such as slab geometry, strand type, number of strands, etc. More research is needed to develop a more comprehensive understanding of this mechanism. A high prestress force, on one hand, benefits flexural behavior. On the other hand, it brings unfavorable effect on the web-shear resistance. In such a case, a high level of prestressing force may be responsible for the brittle failure of deep PCHC units since these members are commonly designed with a higher level of prestressing force as a result of congested strands.

Based on the experimental and numerical findings from this chapter, an improved analytical approach adopting a biaxial failure criterion to account for biaxial stress state of concrete in webs to determine web-shear strength of PCHC slabs was proposed and verified. It will be introduced in the following chapter (Chapter 5).

CHAPTER 5 RESISTANCE OF PCHC SLABS AGAINST WEB-SHEAR FAILURE: ANALYTICAL APPROACH

5.1 Introduction

Experimental results from Chapter 4 show that deep PCHC slabs could fail at imposed loads lower than those predicted by some design codes such as ACI 318, Eurocode 2, and EN 1168. The over-estimation from these codes are due to the three shortcomings:

1. ACI 318 and Eurocode 2 assume that the critical point at which a web-shear crack initiates is close to the neutral axis whereas numerical results presented in Chapter 4 show that for deep PCHC slabs characterized by non-circular voids, the critical point is at the intersection of the web and the bottom flange (Fig. 4.22). This observation agrees well with the studies of Yang (1994) and Brunesi, Bolognini et al. (2015) on web-shear behavior of deep PCHC slabs.
2. As shown in Chapter 4, in longitudinal direction, the critical point is located near the bearing support and within the transmission length region. However, ACI 318 and Eurocode 2 ignore additional shear stresses due to changes of prestressing force within the region.
3. In the traditional design approach (presented in Section 2.2.2.2 and adopted in ACI 318, Eurocode 2 and EN 1168), the maximum principal stress theory is used, in which web-shear failure occurs once the maximum principal tensile stress in concrete webs exceeds the tensile strength of concrete. However, as shown in Chapter 4, the stress state in concrete webs of PCHC slabs is biaxial principal compression-tension (Fig. 4.10). For concrete under the stress state, due to the presence of the principal compressive stress in a direction perpendicular to the maximum principal stress, the allowable principal tensile stress to reach yield

surface is smaller compared to the tensile strength of concrete (Kupfer and Gerstle 1973).

Yang (1994) proposed an approach to determine web-shear capacity of PCHC slabs considering the actual critical locations and the additional shear stresses due to the transfer of prestressing force in transmission length (addressing the 1st and 2nd issues in Section 5.1). Compared to Eurocode 2, Yang's method provided a much better fit with test results (Pajari 2005) and was adopted in EN-1168:2005+A3 (2011). Nonetheless, Pajari (2005) and Brunesi, Bolognini et al. (2015) showed that Yang's approach was still unconservative for some deep PCHC specimens.

To bridge this gap, a new semi-analytical approach to determine web-shear capacity of PCHC slabs was proposed and is presented in this chapter. Besides considering (1) potential critical locations in vertical and longitudinal directions where web-shear cracks are initiated and (2) the additional shear stresses, which are based on Yang's approach, the model addressed the third shortcoming by adopting a more rational failure criterion from Kupfer and Gerstle (1973) for concrete in biaxial compression-tension state. The proposed model was then verified using test data from 2 slabs failed in web-shear in Series I (Chapter 4) and another 45 PCHC slabs from literature. It is shown that the practice of applying the applicable failure criterion has a significant impact on providing safe shear-strength predictions for deep PCHC slabs. This is because Nguyen, Tan et al. (2019) showed that deep PCHC slabs are commonly designed with a large number of pretensioned strands. The high level of compression force as a result of congested strands reduces the resistance in tension of concrete webs in the direction of maximum principal tensile stress, decreasing web-shear strength of deep PCHC slabs. However, this aspect was not considered in the design codes as well as in Yang's approach.

A review of the traditional approach used in ACI 318 and Eurocode 2 was already presented in Chapter 2 (Section 2.2.2.2), the next section reviews key points in Yang (1994)'s method, followed by the proposed model and verification.

5.2 A review of Yang (1994)'s method

Change in prestress along strands

A parabolic function for prestress of strands along the slab length (Fig. 5.1) given in the Swedish code for concrete structures (1979) and FIP Recommendations (1988) was adopted in Yang's approach as follows:

$$\sigma_p = \begin{cases} 0 & 0 \leq x + L_s \leq 5\phi \\ \left[1 - \left(\frac{L_t + 5\phi - x - L_s}{L_t} \right)^2 \right] \times \sigma_{p0} & 5\phi \leq x + L_s \leq 5\phi + L_t \\ \sigma_{p0} & x + L_s \geq 5\phi + L_t \end{cases} \quad (5.1)$$

Fig. 5.1 Change in prestress along strands (Yang 1994)

where σ_p is the prestress of strands; σ_{p0} is the effective prestress; L_s is the distance between the slab edge and the center of the bearing; ϕ is strand diameter; L_t is the transmission length, taken as 55ϕ for slow release and 60ϕ for rapid release.

Critical point

Fig. 5.2 and Fig. 5.3 show the critical locations in PCHC slabs with circular and non-circular voids, respectively. In vertical direction, the critical point is considered to be close to the neutral axis if the hollow-core section has circular voids (Fig. 5.2). For PCHC slabs with non-circular voids and flat webs, the critical location is at the intersection

between the bottom flange (Fig. 5.3). In longitudinal direction, the critical point is at the intersection of the critical level and the line from the center of bearing support at an angle of 35° .

Fig. 5.2 Location of critical point of PCHC slabs with circular voids (Yang 1994)

Fig. 5.3 Location of critical point of PCHC slabs with non-circular voids (Yang 1994)

Web-shear strength according Yang (1994)

Yang's method was developed with the following assumptions:

1. The vertical normal stresses $\sigma_z = 0$;
2. The bending moment at the critical point $M_y(x_{cp})$ can be taken as follows:

$$M_y \approx R_y x_{cp} \approx V_z x_{cp} \quad (5.2)$$

3. The axial normal stresses σ_x and shear stresses τ_{xz} are given as follows:

$$\sigma_x = -\frac{N_p(x_{cp})}{A} - \frac{M_y(x_{cp}) - N_p(x_{cp})e}{I_y} z \approx -\frac{N_p}{A} + \frac{N_p}{I_y} ez - \frac{V_z}{I_y} x_{cp} z \quad (5.3)$$

$$\tau_{xz} = -\frac{1}{b(z_{cp})} \int_{A_{cp}} \frac{d\sigma}{dx} dA = \frac{1}{b} \int_{A_{cp}} \left[\frac{1}{A} \frac{dN_p}{dx} - \frac{ze}{I_y} \frac{dN_p}{dx} + \frac{z}{I_y} \frac{dM_y}{dx} \right] dA \quad (5.4)$$

$$\rightarrow \tau_{xz} = \frac{1}{b} \left[\left(\frac{A_{cp}}{A} - \frac{S_{cp}e}{I_y} \right) \frac{dN_p}{dx} + \frac{S_{cp}}{I_y} V_z \right] \quad (5.5)$$

where R_y is the reaction force at support; V_z is the shear force; $dN_p(x_{cp})/dx$ is the gradient of the prestressing force of strands at the critical point; $A_{cp} = \int_{z_{cp}}^{z_{top}} \int_{b(z)} dydz$ = area of cross section above the critical point; $S_{cp} = \int_{z_{cp}}^{z_{top}} \int_{b(z)} z dydz$ = first moment of area of the cross section above the critical point; and z_{top} is the z -coordinate of the slab top soffit.

4. The maximum stress theory is adopted, in which shear failure occurs when the principal tensile stress σ_1 in concrete web reaches tensile strength of concrete f_{ct} :

$$f_{ct} = \sigma_1 = \frac{\sigma_x}{2} + \frac{1}{2} \sqrt{\sigma_x^2 + 4\tau_{xz}^2} \quad (5.6)$$

Substituting Eqs. 5.3 and 5.5 into Eq. 5.6, web-shear capacity of PCHC slabs can be calculated using the following formula:

$$\begin{aligned} V_c = V_{z.\max} = \frac{bI_y}{S_{cp}} & \left\{ \frac{b}{2S_{cp}} f_{ct} x_{cp} z_{cp} + \frac{S_{cp}}{b} \left(\frac{e}{I_y} - \frac{A_{cp}}{AS_{cp}} \right) \frac{dN_p}{dx} \right. \\ & + \left[\left(\frac{b}{2S_{cp}} f_{ct} x_{cp} z_{cp} \right)^2 + \left(\frac{e}{I_y} - \frac{A_{cp}}{AS_{cp}} \right) f_{ct} x_{cp} z_{cp} \right. \\ & \left. \left. + \left(\frac{1}{A} - \frac{z_{cp}e}{I_y} \right) N_p f_{ct} + f_{ct}^2 \right]^{1/2} \right\} \quad (5.7) \end{aligned}$$

As shown, Yang (1994) considered additional shear stresses due to the change in prestress of strands within the transmission length region. In addition, the model considered critical locations where web-shear cracks could initiate. However, in Yang's approach, the maximum principal stress theory was used. This could lead to unconservative predictions for web-shear strength of PCHC slabs because the stress state in concrete webs of the slabs is not uniaxial tension but biaxial compression-tension as shown in Fig. 2.11. Therefore, a new approach to determine the web-shear strength of PCHC slabs was proposed and is presented in the next section. In the proposed formula, besides considering the additional shear stresses and feasible locations where web-shear cracks emanate, which is similar to Yang's method, the proposed approach takes into

account the biaxial stress state of the concrete in webs by adopting a more rational failure criterion from Kupfer and Gerstle (1973).

5.3 Web-shear strength of PCHC slabs – Proposed model

The design procedure to determine web-shear capacity of PCHC slabs was proposed with the following assumptions:

1. Shape function of strand prestress and transmission length

For simplicity, a linear instead of parabolic function as shown in Fig. 5.4 for prestress of strands within transmission length was assumed in the proposed approach. The linear shape function is also adopted in the design codes such as ACI 318 (ACI 2014) and Eurocode 2 (EN-1992-1-1 2004).

Fig. 5.4 Prestress of strands along slab length

2. Critical point

Fig. 5.5 shows the crack patterns of the two slabs failed in web-shear mode in Series I specimens. Based on observations from the experimental and numerical studies presented in Chapter 4, in this proposed approach, the line of failure (web-shear crack) is considered to start from the inner edge of the bearing support, forming an inclined line at an angle of $\beta \approx 35^\circ$ with respect to the horizontal axis towards the loading location (Fig. 5.6(a)). The critical locations are presumed to be at the mid-height of PCHC slabs with circular voids (where slab webs have a minimum thickness (Fig. 5.6(b)) and to be

at the intersection of the bottom flange and the web (Fig. 5.6(c)). These assumptions are similar to those in EN 1168 (EN-1168:2005+A3 2011) and Yang's approach.

Fig. 5.5 Crack pattern of test specimens

Fig. 5.6 Critical point location in longitudinal and vertical directions (partly adapted from Yang (1994))

3. Normal and shear stresses

Under the effect of vertical loading and prestressing, vertical normal stress σ_z in PCHC slabs is assumed to be zero. In addition, the bending moment at the critical point can be estimated using Eq. 5.2. The two assumptions were also adopted in the traditional design approach and Yang's method.

The elementary beam theory was used to determine normal and shear stresses at the critical point in the concrete webs. The derivation for these stresses (from Eqs. 5.8 to 5.16) is from Yang (1994). Fig. 5.7 shows a diagram of the horizontal normal stress σ_x due to prestress and bending moment over the slab depth.

Fig. 5.7 Horizontal normal stress due to prestress and bending moment

σ_x at the critical point is calculated as follows:

$$\sigma_x = -\frac{N_p(x_{cp})}{A} - \frac{M_y(x_{cp}) - N_p(x_{cp})e}{I_y} z_{cp} \approx -\frac{N_p}{A} - \frac{V_z}{I_y} x_{cp} z_{cp} + \frac{N_p}{I_y} e z_{cp} \quad (5.8)$$

where x_{cp} is horizontal distance from the center of support to the critical point; z_{cp} is vertical distance from the neutral axis to the critical point; $N_p(x_{cp})$ is the effective prestress force at the critical point; V_z is shear force at the critical point; A is the area of hollow-core cross section; $M_y(x_{cp}) \approx R_y x_{cp} \approx V_z \times x_{cp}$ is the bending moment at the critical point; e is eccentricity of the strand; I_y is the moment inertia of cross section.

For PCHC slabs without upper strands, a free body diagram at the critical point is shown in Fig. 5.8(a).

Fig. 5.8 Free body diagram and stress state at the critical point

From force equilibrium $\sum_{\rightarrow} F_x = 0$, the following formula can be derived:

$$\tau_{xz} b(z_{cp}) dx - \int_{A_{cp}} \sigma dA + \int_{A_{cp}} (\sigma + d\sigma) dA = 0 \quad (5.9)$$

$$\rightarrow \tau_{xz} = -\frac{1}{b(z_{cp})} \int_{A_{cp}} \frac{d\sigma}{dx} dA \quad (5.10)$$

Substituting Eq. 5.8 into Eq. 5.10, the shear stress τ_{xz} can be calculated as follows:

$$\tau_{xz} = -\frac{1}{b(z_{cp})} \int_{A_{cp}} \left[-\frac{1}{A} \frac{dN_p}{dx} - \frac{z}{I_y} \frac{dM_y}{dx} + \frac{ez}{I_y} \frac{dN_p}{dx} \right] dA \quad (5.11)$$

$$\rightarrow \tau_{xz} = \frac{1}{b(z_{cp})} \frac{1}{A} \frac{dN_p}{dx} \int_{A_{cp}} dA + \frac{1}{b(z_{cp})} \frac{V_z}{I_y} \int_{A_{cp}} z dA - \frac{1}{b(z_{cp})} \frac{e}{I_y} \frac{dN_p}{dx} \int_{A_{cp}} z dA \quad (5.12)$$

$$\rightarrow \tau_{xz} = \frac{1}{b(z_{cp})} \frac{dN_p}{dx} \left[\frac{1}{A} \int_{A_{cp}} dA - \frac{e}{I_y} \int_{A_{cp}} z dA \right] + \frac{1}{b(z_{cp})} \frac{V_z}{I_y} \int_{A_{cp}} z dA \quad (5.13)$$

$$\rightarrow \tau_{xz} = \frac{1}{b(z_{cp})} \left[\left(\frac{A_{cp}}{A} - \frac{S_{cp}}{I_y} e \right) \frac{dN_p}{dx} + \frac{S_{cp}}{I_y} V_z \right] \quad (5.14)$$

The principal tensile stress (σ_1) and compressive stress (σ_2) at the critical point can then be calculated through σ_x and τ_{xz} as follows:

$$\sigma_1 = \frac{\sigma_x}{2} + \sqrt{\left(\frac{\sigma_x}{2} \right)^2 + (\tau_{xz})^2} \quad (5.15)$$

$$\text{and } \sigma_2 = \frac{\sigma_x}{2} - \sqrt{\left(\frac{\sigma_x}{2} \right)^2 + (\tau_{xz})^2} \quad (5.16)$$

4. Failure criteria

In the traditional design procedure adopted in current design codes and Yang's approach, web-shear failure takes place if the principal tensile stress σ_1 (Eq. 5.15) reaches tensile strength of concrete. However, this approach is not conservative as the stress state in concrete webs of PCHC slabs is biaxial principal compression-tension as shown in Fig. 4.10. For such a stress state, with the presence of compression forces in a direction perpendicular to the principal tensile stress, resistance in tension of concrete webs in the

direction of principal tensile stress decreases (Nguyen, Tan et al. 2019). Thus, a more rational failure criterion should be considered for conservative predictions. For this reason, a yield surface for concrete in biaxial stress state proposed by Kupfer and Gerstle (1973) was adopted in this study. Fig. 5.9 show Kupfer and Gerstle (1973)'s yield surface for concrete in biaxial stress state.

Fig. 5.9 Yield surface of concrete in biaxial stress state from Kupfer and Gerstle (1973)

In this study, σ_1 (positive sign) is the principal tensile stress and σ_2 (negative sign) is the principal compressive stress. Therefore, the yield function for concrete in biaxial compression-tension can be re-expressed as:

$$\frac{\sigma_1}{f_{ct}} = 1 + 0.8 \frac{\sigma_2}{f_c} \quad (5.17)$$

where f_c and f_{ct} are compressive strength and tensile strength of concrete, respectively. Basically, when Eq. 5.17 is adopted, compressive stress (σ_2) at the critical point affects allowable maximum principal stress (σ_1) and web-shear capacity of PCHC slabs.

5. Web-shear strength of PCHC slabs

The problem of defining web-shear strength $V_c = V_{z,max}$ can be summarized as follows:

From classical beam theory:

$$\sigma_x = -\frac{N_p(x_{cp})}{A} - \frac{M_y(x_{cp}) - N_p(x_{cp})e}{I_y} z_{cp} \approx -\frac{N_p}{A} - \frac{V_z}{I_y} x_{cp} z_{cp} + \frac{N_p}{I_y} e z_{cp} \quad (\text{Eq. 5.8})$$

$$\tau_{xz} = \frac{1}{b(z_{cp})} \left[\left(\frac{A_{cp}}{A} - \frac{S_{cp}}{I_y} e \right) \frac{dN_p}{dx} + \frac{S_{cp}}{I_y} V_z \right] \quad (\text{Eq. 5.14})$$

$$\sigma_1 = \frac{\sigma_x}{2} + \sqrt{\left(\frac{\sigma_x}{2} \right)^2 + (\tau_{xz})^2} \quad (\text{Eq. 5.15})$$

$$\sigma_2 = \frac{\sigma_x}{2} - \sqrt{\left(\frac{\sigma_x}{2} \right)^2 + (\tau_{xz})^2} \quad (\text{Eq. 5.16})$$

From the adopted yield surface from Kupfer and Gerstle (1973):

$$\frac{\sigma_1}{f_c} = 1 + 0.8 \frac{\sigma_2}{f_c} \quad (\text{Eq. 5.17})$$

Find $V_c = V_{z,max}(N_p, A, I_y, x_{cp}, e, b(z_{cp}), \frac{dN_p}{dx}, A_{cp}, S_{cp}, f_c, f_{ct})?$

In the problem, V_z appears in both Eqs. 5.8 and 5.14, making the solving procedure rather complicated. More importantly, it was not feasible to obtain V_z in a simplified and explicit formula which is convenient for engineers. Thus, for simplicity, the two terms

$\left(-\frac{M_y(x_{cp})}{I_y} z_{cp} + \frac{N_p(x_{cp})e}{I_y} z_{cp} \right)$ in calculating normal stress σ_x in Eq. 5.8 were ignored.

This assumption is reasonable and was adopted in ACI 318 and Eurocode 2 (shown in Section 2.2.2.2). This is because bending moment and effective prestressing force at the critical point (near bearing support) are small. In addition, as shown in Fig. 5.7, in vertical direction (z axis), normal stress σ_x at the critical point (near neutral axis) due to bending and eccentricity of prestress is small compared to normal stresses in the top and bottom concrete fibers. Moreover, the two components have opposite signs (Fig. 5.7), making

the sum small. When the terms of $\left(-\frac{M_y(x_{cp})}{I_y} z + \frac{N_p(x_{cp})e}{I_y} z \right)$ are ignored, Eq. 5.8

becomes:

$$\sigma_x = -\frac{N_p(x_{cp})}{A} - \frac{M_y(x_{cp}) - N_p(x_{cp})e}{I_y} z_{cp} \approx -\frac{N_p}{A} \quad (5.18)$$

From Eq. 5.14, V_z can be expressed as follows:

$$V_z = \left[\tau_{xz} b(z_{cp}) - \left(\frac{A_{cp}}{A} - \frac{S_{cp}}{I_y} e \right) \frac{dN_p}{dx} \right] \frac{I_y}{S_{cp}} \quad (5.19)$$

Let $B = \left(\frac{A_{cp}}{A} - \frac{S_{cp}}{I_y} e \right) \frac{dN_p}{dx}$, Eq. 5.19 embodies the form of:

$$V_z = \left[\tau_{xz} b(z_{cp}) - B \right] \frac{I_y}{S_{cp}} \quad (5.20)$$

To determine $V_{Rd,c} = V_{z,max}$, the maximum shear stress at the critical point τ_{xz} must be determined. τ_{xz} can be defined using Eqs. 5.15, 5.16, and 5.17 and the solution of τ_{xz} is a function of f_c, f_{ct} and normal stress σ_x . MATLAB (2017) was employed to solve the problem. Codes and solutions are shown in Appendix B1. From MATLAB program, maximum shear stress τ_{xz} can be taken as follows:

$$\tau_{xz} = 2.236 \frac{\sqrt{f_c f_{ct} (5f_c + 4\sigma_x)(f_{ct} - \sigma_x)}}{(5f_c + 4f_{ct})} \quad (5.21)$$

As $N_p, A, I_y, x_{cp}, e, b(z_{cp}), \frac{dN_p}{dx}, A_{cp}, S_{cp}, f_c, f_{ct}$ are known values, $V_{z,max}$ can then be determined.

The procedure to calculate web-shear strength of PCHC slabs can be summarized as follows:

$$V_{Rd,c} = V_{z,max} = \left[\tau_{xz} b_w - B \right] \frac{I_y}{S_{cp}} \quad (\text{Eq. 5.20})$$

where:

$V_{Rd,c}$ is web-shear capacity of PCHC slabs;

$$B = \left(\frac{A_{cp}}{A} - \frac{S_{cp}}{I_y} e \right) \frac{dN_p}{dx};$$

$\sigma_x = -\frac{N_p}{A}$ is horizontal normal stress at the critical point;

$\tau_{xz} = 2.236 \frac{\sqrt{f_c f_{ct} (5f_c + 4\sigma_x)(f_{ct} - \sigma_x)}}{(5f_c + 4f_{ct})}$ is shear stress at the critical

point at failure;

b_w is minimum web thickness (thickness at the critical point);

$A_{cp} = \int_{z_{cp}}^{z_{top}} \int_{b(z)} dy dz$ is cross-section area above the critical level;

$S_{cp} = \int_{z_{cp}}^{z_{top}} \int_{b(z)} z dy dz$ is first moment of area above the critical level;

N_p is effective prestressing force of strands at the critical point;

A is area of cross-section;

z_{cp} is z-coordinate of the critical point (Fig. 5.6(b)&(c));

I_y is moment inertia of the entire section;

e is eccentricity of strands;

f_c is compressive strength of concrete;

f_{ct} is tensile strength of concrete;

$\frac{dN_p}{dx}$ is gradient of prestressing force at critical point, depending on prestress-

distribution function of strands within transmission length. In the proposed method, a linear distribution is assumed. The gradient of prestressing force

at critical point can be calculated as $\frac{N_p(x=L_t)}{L_t}$.

It should be noted that Eq. 5.20 is developed for PCHC slabs without prestressing strands above the critical level. In addition, it is applicable for hollow-core slabs with one layer of prestressing strands, which is the most popular design in practice. A similar procedure can be applied to develop a formula for web-shear strength of PCHC slabs with more than one layer of prestressing strands.

5.4 Evaluation of the proposed formula (Eq. 5.20)

The proposed model was verified using a database consisting of 47 PCHC slabs that failed in web-shear mode from this study and published literature. The database included 45 PCHC slabs compiled by Pajari (2005) and the two slabs from Series I of this study (HC2.400 and HC3.500). It should be noted that Pajari (2005)'s database included test data of a total of 49 slabs. However, 4 slabs were excluded from this verification as they contained compression prestressing strands, which is not within the scope of Eq. 5.20. In the end, a database of 47 PCHC slabs summarized in Table 5.1 was used to verify the proposed model. Detailed information of the 47 slabs in the database is given in Appendix B2.

Table 5.1 Summary of the database used to verify the proposed formula

Slab depth (mm)	No. of test	Type of core	a/d	Failure mode
200	4	Circular	3.4 to 5.7	Web-shear
265	18	Circular	3.0 to 5.2	Web-shear
320	4	Circular	2.8 to 3.6	Web-shear
320	4	Non-circular	2.7 to 4.2	Web-shear
370	2	Non-circular	2.9 to 3.5	Web-shear
400	8	Non-circular	2.3 to 4.0	Web-shear
500	7	Non-circular	1.8 to 3.6	Web-shear
Total	47	Circular & non-circular	1.8 to 5.7	Web-shear

Fig. 5.10 and Fig. 5.11 plot experimental shear strengths against predicted shear strengths using ACI 318-14, Eurocode 2, Yang's method, and the author's proposed model. In the figures, predictions from EN 1168 were not included as they are similar to those in Yang's model. While the mean strengths of concrete including the mean compressive and tensile strengths were used in Fig. 5.10, characteristic strengths ($f_{ck} = f_{cm} - 8$; $f_{ctk} = 0.7f_{ctm}$ as specified in Eurocode 2) were used in predicted strengths as shown in Fig. 5.11. It should be noted that in all the calculations, a ratio of 15% for prestress loss was assumed. This assumption is conservative based on calculated values of loss of

prestressing force for the four PCHC slabs tested in Series I (shown in Table 4-2, Chapter 4) and was also adopted in previous studies (Pajari 2005, Brunesi, Bolognini et al. 2015). In addition, safety factors including material partial safety and strength reduction factors were taken as unity in all calculations.

a) ACI 318-14

b) Eurocode 2

c) Yang's method

d) Proposed model

Fig. 5.10 Shear strengths from experiments vs. predicted shear strengths using mean concrete strength following to (a) ACI 318-14; (b) Eurocode 2; (c) Yang's method; (d) Proposed model

a) ACI 318-14

b) Eurocode 2

c) Yang's method

d) Proposed model

Fig. 5.11 Shear strengths from experiments vs. predicted shear strengths using characteristic concrete strength following to (a) ACI 318-14; (b) Eurocode 2; (c) Yang's method; (d) Proposed model

When the mean strength values of concrete were used, Fig. 5.10 shows that ACI 318-14 provided overly conservative predictions for almost all the slabs in the database. This is partly because a reduction factor of 0.5 was applied to the web-shear capacity of all the slabs with depth more than 12.5 in (318 mm) as required by ACI 318-14. On the other

hand, Eurocode 2 provided unsafe predictions for web-shear strength for the majority of the slabs in the database with a mean ratio of V_{exp}/V_{pre} of 0.83. In contrast, Yang (1994) and the proposed model provided more accurate predictions for web-shear strength of PCHC slabs with respective mean ratios of V_{exp}/V_{pre} of 0.92 (CoV of 0.16) and 0.99 (CoV of 0.18). It is shown that there is an improvement for the author's model over Yang's model in terms of conservative prediction and accuracy.

When characteristic strength values of concrete were used, Fig. 5.11 shows that the same trends were still observed in ACI 318-14 and Eurocode 2 predictions. While ACI 318-14 was overly conservative for predicting shear strength of hollow-core slabs, Eurocode 2 still provided unsafe predictions for about one third of the tested slabs. Therefore, it is recommended that Eurocode 2 should not be used in designing PCHC slabs against web-shear failure. When characteristic strengths were used, the predictions shifted from the slightly unsafe side to the safe side in both Yang's and the author's proposed models. However, while Yang's method was still unconservative for 6 out of the 47 slabs in the database (12%) including 4 deep PCHC slabs, the unsafe cases were reduced from 6 to 2 from the proposed model (4%). Moreover, all predictions for deep PCHC slabs were on the safe side when the proposed model was used. Fig. 5.12 shows detailed ratios of V_{exp}/V_{pre} using the proposed model for each slab depth.

a) V_{exp}/V_{pre} vs. slab depth (using mean strength of concrete)

b) V_{exp}/V_{pre} vs. slab depth (using characteristic strength of concrete)

Fig. 5.12 Evaluation of the proposed formula (Eq. 5.20)

With the mean ratios of experimental shear strength to predicted shear strength of 0.99 (CoV of 0.18) and 1.31 (CoV of 0.16) when mean and characteristic strengths of concrete were used, respectively, it is shown that the proposed formula (Eq. 5.20) was capable of predicting web-shear strength of PCHC slabs. Moreover, the verification shown in Fig. 5.10 to Fig. 5.12 demonstrates that among the four considered models, the proposed formula provided the most reasonable and safe predictions for web-shear strength of hollow-core slabs and therefore can be adopted in design codes.

5.5 Summary and conclusions

A semi-analytical procedure to determine web-shear strength of PCHC slabs was proposed and verified in this chapter. The development of the model is based on the fact that some design codes including ACI 318 and Eurocode 2, and Yang (1994)'s model provide unsafe predictions for web-shear strength of some deep PCHC slabs as shown in previous studies (Walraven and Mercx 1983, Pajari 2005, Hawkins and Ghosh 2006, Palmer and Schultz 2011, Brunesi, Bolognini et al. 2015, Nguyen, Tan et al. 2019). The reason partly lies on the fact that ACI 318, Eurocode 2, and Yang (1994) used the maximum principal stress theory as the failure criterion assuming that web-shear failure

occurs when the principal tensile stress in the concrete web reaches tensile strength of concrete. However, it is shown that the stress state in concrete webs of PCHC slabs is biaxial principal compression-tension (Nguyen, Tan et al. 2019). In such a stress state, the presence of principal compression reduces the allowable principal tensile stress (Fig. 5.9).

In the proposed model, a more rational failure criterion for concrete in biaxial principal compression-tension stress state from Kupfer and Gerstle (1973) was adopted. The verification using a database of 47 PCHC slabs shows that ACI 318-14 was overly conservative for the web-shear resistance due to a factor of 0.5 applied to all deep PCHC slabs with thickness exceeding 318 mm. On the other hand, Eurocode 2 overestimated the web-shear strength of PCHC slabs and should not be used. Yang (1994)'s method gave a much better fit with test results. However, his approach was still unconservative for some deep PCHC slabs even when characteristic strengths of concrete were used. Due to an improvement in theory compared to Yang's method, the author's model provided the most reasonable and accurate predictions for web-shear capacity of PCHC slabs. Therefore, it is recommended to replace the current models in Eurocode 2 and ACI 318 for designing PCHC slabs against web-shear failure.

CHAPTER 6 EXPERIMENTAL AND NUMERICAL INVESTIGATIONS ON WEB-SHEAR BEHAVIOR OF DEEP PCHC SLABS UNDER FIRE CONDITIONS (SERIES II)

6.1 Introduction

As mentioned in Chapter 2, under elevated temperatures, web-shear capacity of PCHC slabs significantly decreases due to (1) temperature-induced losses in strength of concrete and prestressing steels and (2) temperature-induced tensile stresses in concrete webs. The latter is more pronounced for deeper units. However, most fire shear tests in the literature were conducted on shallow PCHC slabs with a depth smaller than 300 mm. To date, there were only four fire shear tests performed on deep PCHC slabs with a depth of 400 mm. As a result, these tests did not provide adequate information for an in-depth study on shear response of PCHC slabs subjected to fire. Consequently, mechanism of fire-exposed PCHC slabs failed in web-shear has not been well established.

To bridge the gap in knowledge, Series II of the test program was conducted. Two types of fire test, viz. heat transfer (one specimen) and structural fire (four specimens) tests, were carried out. The effect of slab thickness and load level on web-shear behavior of deep PCHC slabs were investigated. Data obtained from this series were analyzed to study web-shear failure mechanism of deep PCHC slabs under fire conditions. In addition, they were used to verify two FE models that were developed to simulate thermal and structural responses of deep PCHC slabs subjected to fire. The FE models were then used to extend the existing database of deep PCHC slabs tested under fire conditions. Furthermore, the test data were used to evaluate the validity of the shear formula in EN 1168 for PCHC slabs in the event of fire. Details of the experimental program, test results, numerical studies and code comparison are discussed in this chapter.

6.2 Experimental program

As mentioned in Section 3.2.1, two types of fire test were carried out in Series II, i.e. heat transfer (one test) and structural fire tests (four tests). Table 6.1 summarizes the key information of the five tests.

Table 6.1 Summary of fire tests

Slab ID.	Depth (mm)	Axis distance (mm)	Shear span (mm)	Ambient capacity* (kN)	Applied load (kN) (load level (%))	Type of test
H.HC.400	400	70	n.a	na	0 (0)	HT
F.HC1.320	320	40	800	186	74 (40)	ST
F.HC2.400	400	45	800	511	204 (40)	ST
F.HC3.500	500	52.5	800	1160	464 (40)	ST
F.HC4.500	500	52.5	800	1160	290 (25)	ST

* Defined by tests by Nguyen, Tan et al. (2019); HT = Heat transfer; ST = Structural test

6.2.1 Heat transfer test

6.2.1.1 Specimen properties

Table 6.2 presents the geometric properties of H.HC.400. The average concrete compressive strength measured on three concrete cubes of 150 mm × 150 mm at 28 days was 65.3 MPa, which corresponded to 54 MPa mean cylinder strength based on Table 3.1 – Eurocode 2 (EN-1992-1-1 2004). The specimen was considered as normal-strength concrete and spalling of concrete was not expected. The heat transfer test was conducted after 248 days of casting. As a result, the moisture content of the specimen was at a stable level.

Table 6.2 Geometric properties of H.HC.400

	H.HC.400
Cross-section area, A	0.265 (m ²)
Cross-section height, h	400 (mm)
Cross-section width, b	1.2 (m)
Specimen length, L	2.2 (m)
Total web width, b_w	350 (5 webs of 70 mm) (mm)
Top flange min. thickness	60 (mm)
Bottom flange min. thickness	60 (mm)

6.2.1.2 Instrumentation

Since the objective of the heat transfer test was to obtain temperature profile, no external load was applied on H.HC.400. The instrumentation included only type K thermocouples installed over the slab depth through drilled holes from the slab soffit. Once the thermocouples had been embedded, the holes were filled with cement paste. The thermocouples were embedded at two different sections along the specimen length, namely, Section A, and Section B. Section A was at the center line of the slab while Section B was 600 mm from one edge of the specimen. Fig. 6.1 shows the thermocouple arrangement in the heat transfer test.

a) Section A and Section B in longitudinal direction

b) Thermocouples in vertical direction

Fig. 6.1 Thermocouple arrangement

6.2.1.3 Test setup & fire scenario

Fig. 6.2(a) shows the overall setup for the heat transfer test. The specimen was placed on top of an ISO electrical furnace with interior a dimension of 0.7 m wide by 1.7 m long by 0.75 deep and was simply supported at the two ends by two A-frames. No axial restraint was applied to the slab during the test. Also, the two ends of the longitudinal cores were also filled by insulation material (thermal wool). The specimen was heated for three hours following the ISO-834 fire curve. Fig. 6.2(b) shows the standard fire curve ISO-834 together with the measured temperature inside the furnace. It illustrates that the electrical furnace could be controlled to follow the standard fire condition for a duration of up to three hours.

Fig. 6.2 Heat transfer test setup & Fire scenario

6.2.2 Structural fire shear tests

Geometric and material properties of the four slabs in the structural fire tests were given in Sections 3.2.2.1 and 3.2.2.2, respectively. This section presents their detailed test setup, instrumentation, testing procedure and heating curves.

6.2.2.1 Test setup & Instrumentation

The test setup for structural fire tests is shown in Fig. 6.3. All the specimens were placed on top of an electric furnace whose interior dimensions were 3 m wide, 3 m long, and 0.75 m deep, and they were simply supported at their two ends by two A-frames without any axial restraint. They were loaded by two line loads that were symmetrically placed about the slab center. The shear spans were 800 mm. In addition, since the width of all the hollow-core specimens was only 1.2 m while the width of the furnace was 3 m, steel panels and thermal wool were placed upon the open parts of the furnace to prevent heat loss to the surrounding environment.

Fig. 6.3 Setup of the fire shear tests

The instrumentation in the structural fire tests included load cells, linear variable displacement transducers (LVDTs) and type K thermocouples (T). There were three load cells (LC1 to LC3) arranged on top of a loading beam and two spreader beams to measure loads transferred from a 100-ton hydraulic jack to the specimen. Eight LVDTs (LVDTs 1 to 8) were placed on top of each specimen to monitor vertical displacements at the mid-span, locations of applied loads and the two supports. Additionally, four LVDTs (LVDTs 9 to 12) were used to measure longitudinal expansion at the two ends of the specimens during heating. In addition, thermocouples were embedded in the specimens to track temperature development during the fire tests. Fig. 6.4 generally shows the load cells and LVDT arrangement whereas locations of the thermocouples are shown in Fig. 6.5.

Fig. 6.4 Load cell and LVDT arrangement

Fig. 6.5 Thermocouple arrangement

6.2.2.2 Testing procedure and heating program

The structural fire tests in this study followed a transient-state regime, in which vertical loads were introduced first and heat was subsequently applied. To study the effect of load level on shear behavior, two different load levels were applied. While F.HC1.320, F.HC2.400, and F.HC3.500 were loaded up to 40% of their ambient capacity (defined by tests, given in Nguyen, Tan et al. (2019) and shown in Table 6.1), F.HC4.500 was only loaded up to 25%. The four hollow-core slabs were then heated up to failure while the imposed loads were maintained constant during the tests. All four of the specimens were programed to follow a fire curve which increased linearly from ambient to 1000°C at 60 min and remained at this temperature until the end of the test. It is worth noting that the selected heating curve was a non-standard fire curve due to limitations of the larger electrical heating furnace. Thus, the results from the structural fire tests presented in this

chapter could only be used to study shear behavior of deep PCHC slabs under elevated temperatures; Their fire resistance under the standard ISO 834 fire curve could only be obtained numerically through verified finite element models. Fig. 6.6 shows the electric heating curve used in this study together with the measured furnace temperature with time. It should be noted that the structural fire tests began with F.HC2.400 and due to a problem of the furnace, the specimen was exposed to a lower heating curve compared to the expected one (Fig. 6.6). Subsequently, the problem was fixed, and the latter three fire tests followed the intended heating regime.

Fig. 6.6 Electric heating curve and measured furnace temperatures

6.3 Test results and discussions

6.3.1 Heat transfer test (H.HC.400)

Since this was a heat transfer test, there were no loads applied. Results from the experiment will therefore be discussed in terms of thermal response and damage patterns observed in the test.

6.3.1.1 Development of temperature

Temperature profiles recorded by the thermocouples (Fig. 6.1(b)) are plotted in Fig. 6.7. It should be noted that for cross sections with more than one thermocouple, an average value was taken. Data from thermocouples showed reasonable results since the cross-

sectional temperatures in the concrete decreased as the distance from the fire-exposed soffit increased, due to low conductivity and high specific heat capacity of concrete that delayed the heat transfer over the cross section.

Fig. 6.7 Development of temperature over the cross-section of H.HC.400

6.3.1.2 Damage patterns

During the first 25 min of fire exposure, no cracks were observed. However, when the furnace temperature reached about 830°C (at 28 min of fire exposure), longitudinal cracks started occurring at the top and the bottom of the hollow cores, emanating from the two slab edges (Fig. 6.8) due to thermal expansion of concrete in transverse direction. As the temperature increased, these cracks propagated towards the slab center, resulting in longitudinal cracks along the longitudinal voids as shown in Fig. 6.8. After 100 min of fire exposure, when the furnace temperature reached 1000°C, one longitudinal crack at an inner core propagated through the whole slab length (Fig. 6.9(a)). At this moment, the slab no longer complied with “criterion E” or integrity, based on Eurocode 2 Part 1-2 (EN-1992-1-2 2004). After 120 min of fire exposure, the existing cracks became wider, but no new cracks were observed. The test ended after 3h of heating. It is noteworthy that apart from the longitudinal cracks, spalling was not observed during the fire exposure.

Fig. 6.8 The development of longitudinal cracks

Fig. 6.9 Longitudinal cracks at the top and bottom of H.HC.400

6.3.2 Structural fire shear tests – Test results

Table 6.3 summarizes results from the structural fire tests. Discussions of test results will be focused on thermal response, structural behavior, crack pattern and failure mode.

Table 6.3 Test results

Slab ID.	a/d^*	Applied load (kN)	Furnace temperature at failure ($^{\circ}\text{C}$)	Failure time (min)	Maximum mid-span deflection (mm)	Failure mode
F.HC1.320	2.86	74	944	57	57.8	WS & A**
F.HC2.400	2.25	204	803	118	34.4	Web-shear
F.HC3.500	1.79	464	854	48	14.2	Web-shear
F.HC4.500	1.79	290	881	52	16.9	Web-shear

* a/d : Shear span-to-effective depth ratio; **WS & A = Web-shear and Anchorage

6.3.2.1 Thermal response

Temperature profiles over the cross section of the four slabs measured by the embedded thermocouples (recorded up to failure) are plotted in Fig. 6.10. In the figure, x refers to the vertical distance from the bottom soffit (the fire-exposed surface) to the measured section in mm. Similar to thermocouple readings from the heat transfer test H.HC.400, thermocouple data from the structural fire tests also showed an expected pattern; cross-sectional temperature reduced with increasing x from the heated soffit. In addition, it was observed that at a certain distance x from the slab soffit, there was a sudden increase in temperature from 40 to 50 min of fire exposure (as evident in Fig. 6.10(a) to (d)). Such a sudden increase in temperature was probably attributed to longitudinal cracks formed at the bottom slab along the longitudinal voids, causing an increase in temperature in regions nearby.

a) F.HC1.320

b) F.HC2.400

c) F.HC3.500

d) F.HC4.500

Fig. 6.10 Temperature development over the cross sections

The temperature histories obtained from thermocouples also indicated that temperature gradually increased from ambient to 100°C. After this, a temperature plateau was reached, which started from around 42 min of fire exposure. This plateau was caused by the migration of free water inside the concrete, in the upward direction from the fire-exposed soffit to the slab top surface. As the temperature exceeded 100°C, water was transformed from liquid to vapor, resulting in a moisture clog that inhibited temperature increase until the layers had eventually dried out (Nguyen and Tan 2018).

6.3.2.2 Structural responses

It should be noted that for slabs subjected to fire, there are three failure criteria specified in Eurocode 2 Part 1-2 (EN-1992-1-2 2004), i.e. integrity (criterion E), insulation (criterion I), and load-bearing capacity (criterion R). Criterion R is considered through a deflection limit in the fire test. However, as fire shear tests were conducted, failure was defined at the temperature or time when there was a significant drop in load-carrying capacity caused by the formation of shear cracks. At this moment, the specimens were unable to resist the imposed loads from the hydraulic jack.

Time histories of the imposed load and the mid-span displacement from the four fire tests are plotted in Fig. 6.11. It is shown that all four of the specimens lost their bearing capacity before the furnace temperature reached 1000°C. F.HC1.320, F.HC2.400,

F.HC3.500, and F.HC4.500 failed at 944, 849, 854, and 881°C, respectively. As shear cracks occurred near the supports, the specimens could no longer sustain the imposed loads.

Fig. 6.11 Applied load and mid-span deflection vs. time

6.3.2.3 Cracks and failure modes

Failure mode occurred in the tests for each of the four specimens are given in Table 6.3. While F.HC1.320 lost its bearing capacity due to a combination of web-shear and anchorage failure, the other three slabs (F.HC2.400, F.HC3.500, and F.HC4.500) failed due to formation of diagonal tensile cracks in the concrete webs, the so-called web-shear failure. Fig. 6.12 shows the failure modes in the four tests whereas Fig. 6.13 presents typical cracks. Four types of cracks were identified, viz. splitting, longitudinal, thermal

and web-shear cracks. Other than these, no fire-induced spalling was observed in any of the four tests during the heating time.

a) Web-shear and anchorage failure – F.HC1.320

b) Web-shear failure –
F.HC2.400

c) Web-shear failure – F.HC4.500

d) Web-shear failure – F.HC3.500

Fig. 6.12 Failure modes observed in the tests

Fig. 6.13 Four types of cracks observed in the fire tests (the sequence of formation: (a) → (b) → (c) → (d))

Fig. 6.14 Strands slipped inside concrete at early stage of heating

Localized splitting cracks (Fig. 6.13(a)) occurred between the concrete and the strand when furnace temperature reached about 700°C. These cracks occurred due to a high-temperature gradient within the lower part of the section, resulting in incompatible thermal expansion between the two. The verified FE heat transfer model presented in Section 6.4.1 showed that at furnace temperature of 700 °C, average temperature of

concrete below the strand level and temperature of the strand were 190 and 98 °C, respectively. Thermal expansion of concrete and strand could accordingly be calculated using formulas for thermal strain in Eurocode 2 Part 1-2 (EN-1992-1-2 2004) as follows:

For concrete (using siliceous aggregates):

$$\varepsilon_c(\theta) = -1.8 \times 10^{-4} + 9 \times 10^{-6} \theta + 2.3 \times 10^{-11} \theta^3 \quad \text{for } 20^\circ\text{C} \leq \theta \leq 700^\circ\text{C}$$

Thermal strain of prestressing steel:

$$\varepsilon_p(\theta) = -2.016 \times 10^{-4} + 10^{-5} \theta + 0.4 \times 10^{-8} \theta^2 \quad \text{for } 20^\circ\text{C} \leq \theta \leq 1200^\circ\text{C}$$

For a temperature of 190 °C for concrete and 98 °C for strand, thermal elongation of the concrete and the strand was 3.5 and 1.7 mm, respectively. Such a discrepancy in thermal expansion between the concrete and the strand induced shear stresses between them, promoting splitting cracks to occur. The difference in thermal expansion of the two was evident from the observed slippage of the strands at the early stage of fire exposure as shown in Fig. 6.14. In general, the localized cracks initiated at the two slab ends and then propagated into longitudinal cracks along the strand length as shown in Fig. 6.13(a).

On the other hand, longitudinal cracks (Fig. 6.13(b)) occurred at the top and bottom surfaces of hollow cores when the furnace temperature reached about 800°C due to thermal expansion of concrete in transverse direction. These cracks were first generated at the two supporting edges. Due to elevated temperatures and an absence of steel meshes at both top and bottom flanges of the hollow-core slabs, these cracks propagated towards the slab center, resulting in full-length longitudinal cracks as shown in Fig. 6.13(b). Furthermore, vertical thermal cracks occurred due to a temperature gradient across the section of the specimen, inducing tensile stresses in concrete webs and forming cracks along the concrete webs.

Finally, web-shear cracks occurred, starting in concrete webs in a region between the support and applied-load locations due to a combination of thermal stress and stresses due to prestressing force and applied load in the concrete webs exceeding the tensile strength of concrete. The cracks in concrete webs, once occurred, immediately

propagated towards the applied load and the support regions, forming diagonal web-shear cracks, resulting in a significant drop in load-bearing capacity and causing a brittle failure.

6.4 Numerical analyses

In this study, two 3-D FE models were sequentially developed, viz. heat transfer and mechanical models to respectively investigate thermal and structural responses of PCHC slabs subjected to fire. First, the heat transfer model using ABAQUS/Heat-transfer (Simulia 2014) was built to obtain temperature evolution inside PCHC slabs when exposed to fire. The temperature field determined from the heat transfer model was subsequently mapped onto the mechanical model which was developed using ABAQUS/Explicit (Simulia 2014) for structural analysis. The heat transfer model was verified using temperature data under both ISO and non-ISO fire obtained in the five tests. Additionally, the mechanical FE model was verified against the test data in terms of load and deflection histories, as well as damage and failure modes from the four structural fire tests. Subsequently, the verified mechanical model was used to determine the fire resistance of the PCHC slabs subjected to the ISO-834 fire, obviating the limitation of the furnace employed in the experimental program.

6.4.1 Heat transfer model

In the heat transfer model, concrete was simulated using 8-node linear heat transfer brick elements. The material and geometric properties of H.HC.400 were adopted to develop the model. A mesh convergence study was performed with three mesh sizes of 20, 30, and 40 mm. It was shown that all the three mesh sizes were suitable to simulate the thermal response of H.HC.400. Thus, the mesh size of 40 mm was chosen to reduce computational time.

Thermal interaction

The slab was modeled in a way that it interacted with the fire at the bottom surface and with the natural environment at the top surface. The convection factors for the fire

exposed and unexposed surfaces were $25 \text{ W/m}^2\text{K}$ and $10 \text{ W/m}^2\text{K}$, respectively. The emissivity related to the concrete surface was taken as 0.7. Thermal boundary conditions within the hollow cores were deliberately taken into account. The hollow core was divided into two halves, i.e., the top half and the bottom half that were separated by a horizontal plane cutting through the intersections between the smallest web thickness and the lower flange as illustrated in Fig. 6.15. It was assumed that heat from the bottom core surface was transported to the top core surface by the upward movement of hot air inside the core by convection and radiation (Shakya and Kodur 2015, Aguado, Albero et al. 2016). In addition, the bottom core surface was also affected by the cooler temperature from the top core by radiation. This phenomenon involved absorption, reflection or transmission as the top and bottom core surfaces could radiate back and forth to each other. Inside the hollow cores, the emissivity factor was taken as 0.7 and a convective factor of $25 \text{ W/m}^2\text{K}$ produced the best-fit results.

Fig. 6.15 Thermal interaction

Material properties

In the FE heat-transfer model, temperature-dependent thermal properties of concrete including conductivity, specific heat and density, had to be defined. Thermal conductivity is a property of a material quantifying its ability to conduct heat. It is described as the quantity of heat transferred through a unit thickness in the direction perpendicular to the exposed surface due to a unit temperature gradient. Specific heat, on the other hand, is the amount of heat per unit mass needed to raise the temperature by one unit. Such thermal properties are not constant but vary with temperature. In this

study, the thermal properties of concrete were assumed following the values in Eurocode 2 (EN-1992-1-2 2004), as shown in Fig. 6.16.

Fig. 6.16 Thermal properties of concrete based on Eurocode 2 (EN-1992-1-2 2004)

The thermal properties of longitudinal steel could be ignored since they had very little effect on temperature development in the PCHC slabs. Moreover, the prestressing steel ratios in the PCHC slabs were very low, varying from 0.15% (F.HC1.320) to 0.47% (F.HC2.400). Therefore, strand temperature was assumed to be equal to the concrete temperature at the same level from the slab bottom soffit (Aguado, Alberio et al. 2016, Shakya 2016, Kodur and Shakya 2017).

Model validation

The heat transfer model was also developed for the other four specimens in the structural fire tests using the same techniques. The model was verified by comparing its predicted temperature and the measured temperature developed in the five specimens from the fire tests (shown in Fig. 6.7 and Fig. 6.10). While Fig. 6.17 presents the predicted temperature contour of H.HC4.400 after 3h of fire exposure, Fig. 6.18 representatively shows temperature comparisons between the tests and the FE models for H.HC.400 and F.HC2.400, which had the longest heating duration and the highest recorded temperatures (up to 3h and 2h, respectively). Fig. 6.18 shows good correlation between the two sets of temperature, illustrating that the developed heat transfer model was

capable of tracing temperature development in deep PCHC slabs exposed to both ISO and NON-ISO fires. Therefore, temperature profiles from the FE heat-transfer model were reliable and could be applied to the mechanical model for subsequent structure-fire-load analysis.

Fig. 6.17 Temperature distribution at 3h of fire exposure – H.HC.400

Fig. 6.18 Temperature comparison between the test and the FE heat transfer model

6.4.2 FE mechanical model

Modeling technique

The FE mechanical model using ABAQUS/Explicit (Simulia 2014) was developed for structural analyses of PCHC slabs subjected to fire, with an emphasis on shear behavior. The concrete mesh discretization in the mechanical model had to be identical to that used in the heat transfer model, to allow the temperature at each node from the heat transfer model to be mapped onto those for the mechanical model. Concrete was modeled using 8-node linear brick elements with a reduced integration scheme. Steel strands were

simulated using T3D2 truss elements (2-node linear displacement) and embedded into the concrete assuming perfect bonding. Prestressing loss and transmission length of the strands (calculated using Eurocode 2 (EN-1992-1-1 2004) and shown in Table 6.4) were adopted in the numerical model. The strands were simulated to be stress-free at their two ends and to attain their full effective stress (strand stress after prestressing loss) at the end of their transmission length, following a linear-stress-distribution assumption. A mesh size of 10 mm was applied for the strand while a concrete mesh size of 40 mm was used as mentioned in Section 6.4.1.

Table 6.4 Calculated transmission length and prestressing losses based on Eurocode 2

Slab ID.	Strand diameter (mm)	Transmission length	Prestressing loss (%)	Effective stress (MPa)
F.HC1.320	9.6	313	8.6	1051
F.HC2.400	9.6/12.9	302/406	11.4	1019
F.HC3.500	12.9	410	12.4	1007
F.HC4.500	9.6	313	12.4	1007

Material model

The constitutive model for prestressing strands under elevated temperatures specified in EN-1992-1-2 (2004) was adopted to represent temperature-dependent stress-strain behavior. The *Concrete damage plasticity model* (the CDP model) in Abaqus/Explicit (2014) was selected to simulate nonlinear behavior of concrete.

In the CDP model, concrete response under uniaxial stress state must be defined. The stress-strain curve of concrete in compression with a linear descending branch and strength reduction factors accounting for temperature effects complied with EN-1992-1-2 (2004). The uniaxial tensile behavior of concrete, on the other hand, was described with a stress-crack opening relation to avoid difficulties in convergence due to mesh sensitivity (Demir, Öztürk et al. 2016). The stress-crack opening function proposed by Hordijk (1992) and the Eurocode concrete tensile strength reduction factors due to temperature were adopted in this study. It should be noted that concrete fracture energy

in tension (the area below the stress-crack opening curve) was determined using the Model Code (FIB 2010) and assumed to be unchanged during the heating time. Such an assumption was made as there are limited test data on the fracture energy of concrete as functions of temperature. In addition, existing test data did not show a clear trend describing the effect of temperature on concrete fracture energy (Gao, Dai et al. 2013). Consequently, the ultimate crack opening under elevated temperatures increased as tensile strength of concrete decreased. Fig. 6.19(a) and (b) respectively show concrete constitutive model for F.HC1.320 in compression and in tension. Similar constitutive models were developed for F.HC2.400, F.HC3.500 and F.HC4.500 based on their properties defined by material tests as shown in Table 3.7 and Table 3.8. It is noteworthy that for the CDP model, it was only necessary to define the stress-inelastic strain/crack opening constituent.

a) Compressive behavior

b) Tensile behavior

Fig. 6.19 Constitutive model for concrete (F.HC1.320)

The curves in Fig. 6.19 describe the behavior of concrete in a uniaxial stress state. The behavior of concrete in bi-axial and tri-axial stress states in the CDP model is defined by four plastic parameters, i.e. the dilation angle, the ratio of initial biaxial compressive yield stress to initial uniaxial compressive yield stress, the eccentricity and the ratio of the second stress invariant on the tensile meridian. While the values of the last two parameters were taken as 0.1 and 0.667, respectively, as recommended by ABAQUS (Simulia 2014), the dilation angle that controls the change in plastic volume of concrete under pressure is not only a material property but is also affected by structural behavior (Nguyen, Tan et al. 2019). Dilation angle of 38° , within the range of 15° to 56° as used in previous studies (Aguado, Alberio et al. 2016, Demir, Öztürk et al. 2016, Genikomsou and Polak 2017, Nguyen, Tan et al. 2019), reasonably captured behavior of PCHC slabs exposed to fire and was adopted in this investigation. Lastly, the ratio of initial biaxial compressive yield stress to initial uniaxial compressive yield strength was defined to bilinearly increase from 1.16 at ambient temperature to 1.3 at 300°C and finally to 1.7 at 700°C . These values were based on experimental data from Kordina, Ehm et al. (1985).

Model validation

The proposed mechanical model was verified using the experimental results from the four structural fire tests. Displacement-time histories, cracks, failure modes and furnace temperature at failure were compared between the structural fire tests and the FE models. Fig. 6.20 shows a comparison between the recorded and predicted mid-span displacement against time, whereas comparisons of furnace temperature at failure and failure mode between the tests and the FE models are presented in Table 6.5. Good correlation between prediction and measurement in terms of deflection development, failure point (through furnace temperature at failure) and failure mode, was obtained for F.HC1.320, F.HC3.500, F.HC4.500 (Fig. 6.20(a), (c), (d) and Table 6.5). For F.HC2.400, it is shown that the slab failed earlier in the FE model compared to in the test. However, it is worth mentioning that in the test of F.HC2.400, furnace temperature decreased after reaching 854 °C at 65 min due to a problem of the electrical furnace as mentioned in Section 6.2.2.2. Therefore, even fire endurance time of F.HC2.400 in the FE model was shorted compared to that in the experiment, F.HC2.400 in the numerical analysis failed after the furnace temperature reached its maximum of 854 °C, which is similar to F.HC2.400 in the test. Moreover, web-shear failure mode of F.HC2.400 in the test was well predicted in the FE model (Fig. 6.22).

Additionally, as can be seen in Fig. 6.21, the formations of longitudinal, thermal and web-shear cracks observed in the tests were also captured in the FE models in terms of plastic strain (PE). Moreover, the sudden loss of load-bearing resistance due to propagation of diagonal web-shear cracks could be reproduced in the FE models, as shown in Fig. 6.22. The comparisons demonstrated that the FE models could effectively predict the behavior of the PCHC slabs subjected to fire. However, it is noteworthy that the splitting cracks that occurred in the tests could not be clearly shown in the FE models. In addition, anchorage failure in F.HC1.320 could not be simulated. This failure was caused by a gradual loss in bond strength between the concrete and the strands in the tests. However, in the FE models, a perfect bond condition was assumed. Consequently,

the splitting cracks between the concrete and the strands, as well as anchorage failure, could not be simulated in the FE models. This is a drawback of the developed numerical models.

Fig. 6.20 Comparison of displacement development between the tests and the FE models

Table 6.5 Comparison between the tests and the FE models

Slab ID.	a/d	Applied load (kN)	Furnace tempt at failure (°C)			Failure mode	
			Test	FEM	Test/FEM	Test	FEM
F.HC1.320	2.86	74	945	917	1.03	WS & A	WS
F.HC2.400	2.25	204	803	834	0.96	WS	WS
F.HC3.500	1.79	464	854	828	1.03	WS	WS
F.HC4.500	1.75	290	881	841	1.05	WS	WS

* WS & A = Web-shear and Anchorage; WS = web-shear

Fig. 6.21 Comparison of cracks and failure mode between the experiments and the FE models

Fig. 6.22 Failure of F.HC2.400 due to the formation of web-shear cracks in the FE model

6.4.3 Fire resistance of the PCHC slabs under ISO-834 fire curve

The verified FE models were employed to obtain fire resistance of the four specimens in the structural tests in case they are subjected to ISO 834 fire. The actual geometric and material properties, as well as the loading scheme of F.HC1.320, F.HC2.400, and F.HC3.500, and F.HC4.500 were simulated. Numerical results showed that behavior of web-shear failure mode in which the slabs lost their bearing capacity due to formation of web-shear cracks could be observed in all four of the slabs as typically shown in Fig. 6.23. The furnace temperature at failure and the fire resistance of the four slabs are summarized in Table 6.6.

Fig. 6.23 Web-shear failure of PCHC slabs subjected to ISO-834 fire curve

Table 6.6 Summary of numerical results

Slab ID.	Applied load (kN)	Experiments	FEM (exposed to ISO 834 fire)	
		Maximum furnace temperature (°C)	Maximum furnace temperature (°C)	Fire resistance (min)
F.HC1.320	74	945	993	83
F.HC2.400	204	845	856	33
F.HC3.500	464	854	815	24
F.HC4.500	290	881	891	40

It is noteworthy that the axis distances of all the strands in the four slabs were greater than 40 mm, therefore, based on the prescriptive method in Eurocode 2 (EN-1992-1-2 2004), the hollow-core specimens should sustain at least 2h FR. However, numerical results from Table 6.6 show that all the slabs would fail within 90 min of the standard

fire curve, thus displaying premature failure. These numerical findings are in good agreement with experimental results from Fellingner (2004) who conducted 25 fire shear tests on PCHC units, of which four tests were carried out on 400 mm deep hollow core specimens.

Comparing the furnace temperature at failure between the tests using a non-ISO fire curve and the FE models using the ISO-834 fire curve (Table 6.6), it is suggested that fire tests using non-ISO curves can only be used to estimate the web-shear resistance of PCHC slabs exposed to fire, since regardless of the difference in the fire curves, the slabs still failed at very comparable furnace temperatures as shown in Table 6.6. Moreover, considering the results of F.HC1.320, F.HC2.400 and F.HC3.500, which were loaded at the same level of 40% of their ambient capacities, it is evident that fire resistance decreased as slab thickness increased. This behavior was expected, since under fire conditions nonlinear thermal gradient existed across the slab section; thermal gradient was even more pronounced for deeper units, producing additional tensile stress at the concrete webs. Consequently, the web-shear resistance of PCHC slabs decreased as the slab depth increased.

The effect of load level on web-shear resistance of PCHC slabs was evaluated by comparing the fire resistance between F.HC3.500 and F.HC4.500. Although the load level was decreased from 40 to 25%, the 500 mm thick slab only resisted the imposed load for additional 16 min but still failed within 60 min, as shown in Table 6.6. Numerical investigations were extended with two additional cases, i.e. the same 500 mm thick slab was subjected to load levels of 20% and 10%. The results showed that the 500 mm thick slab could achieve 106 and 159 min of fire exposure with these loads, respectively. However, in general, slabs were not subjected to such a low load ratio. Therefore, it is advised to enhance the web-shear capacity of PCHC slabs through using steel fibers. The study of steel-fiber-reinforced PCHC slabs under fire conditions presented in Chapter 8 will help to quantify this approach.

6.5 Shear strengths of the PCHC slabs according to EN 1168 (EN-1168:2005+A3 2011)

With regard to shear and anchorage failures, EN 1168:2005+A3:2011 provides an empirical equation to determine the shear strength of PCHC slabs subjected to fire. It should be noted that EN 1168 is a harmonized European standard, specifically drafted for the design of PCHC slabs. The shear capacity of fire-exposed PCHC slabs according to EN 1168 is given as follows:

$$V_{Rd,c,fi} = [C_{\theta,1} + \alpha_k \times C_{\theta,2}] \times b_w \times d \quad (\text{Sec.G.1.3})$$

where

$$C_{\theta,1} = 0.15 \min(k_p(\theta_p) \sigma_{cp,20^\circ C}; \frac{F_{R,a,fi,p}}{A_c}) \text{ is a coefficient which accounts for}$$

concrete stress under fire conditions;

$$\alpha_k \text{ is a size factor} = 1 + \sqrt{\frac{200}{d}} \leq 2 \text{ where } d \text{ is in mm;}$$

$$C_{\theta,2} = \sqrt[3]{0.58 \times \frac{F_{R,a,fi}}{f_{yk} \times b_w \times d}} f_{c,fi,m} \text{ is a coefficient that accounts for the anchored}$$

longitudinal reinforcement.

To apply this formula, the strand and concrete temperatures at the mid-height of the web must be known. However, strand-temperature data were not available in the tests since the thermocouples were embedded into the specimens through drilled holes as mentioned in Section 6.2.1.2. During the installation, to prevent potential damage, the drill bit used should not come into contact with the strands. Therefore, these temperatures were determined using the verified FE heat transfer model. Concrete temperature at the mid-height of the web, on the other hand, was defined using the experimental data. The strand and concrete temperatures with their respective strength reduction factors according to Eurocode 2 Part 1-2 (EN-1992-1-2 2004) are presented in Table 6.7, whereas shear-strength comparisons between the tests and the EN 1168 predictions are shown in Table

6.8. It should be noted that concrete mean strength including both the compressive and tensile strength was used in the calculations (instead of characteristic strength). In addition, all safety factors were taken as unity.

Table 6.7 Strand and concrete temperatures and their reduction strength factors based on Eurocode 2

Slab ID.	Time at failure (mm)	Strand temperature (°C)	Reduction factor for strand strength	Mid web temperature (°C)
F.HC1.320	57	226	0.83	100
F.HC2.400	118	269	0.77	99
F.HC3.500	48	118	0.97	34
F.HC4.500	52	135	0.95	43

Table 6.8 Shear-strength comparison

Slab ID.	a/d	Time at failure	Experimental failure load (kN)*	EN 1168 failure load (kN)	$V_{test}/V_{predict}$
F.HC1.320	2.86	57	104	70.5	1.48
F.HC2.400	2.25	118	245	144.5	1.70
F.HC3.500	1.79	48	500	166.6	3.00
F.HC4.500	1.79	52	326	163.8	1.99

* Including weight of specimen and loading system

With $V_{test}/V_{predict} > 1$ in all the cases (Table 6.8), it was shown that the shear formula in EN 1168 gives conservative predictions for shear strength of PCHC slabs similar to the ones tested in the experimental program. However, it is worth mentioning that currently, design of fire resistance of deep PCHC slabs is mostly based on the prescriptive method specified in Table 5.8 - Eurocode 2 Part 1-2 (EN-1992-1-2 2004); the method has been shown in Section 6.4.3 to not be conservative for deep PCHC slabs subjected to shear.

Additionally, it is shown that the two 500 mm thick specimens failed at an early stage when the strand and the concrete temperatures had not reached 140°C (Table 6.7). At such temperatures, based on Eurocode 2, concrete and strand strengths were almost the same as those at ambient condition, suggesting that under fire conditions, temperature-induced stresses in concrete due to nonlinear thermal gradient instead of temperature-induced degradation in strength governed the web-shear resistance of deep hollow-core slabs. This observation is supported by numerical results as shown in Fig. 6.24. The figure compares principal-stress distribution inside F.HC3.500 at two different times: at $t = 0$ (just before heat was applied) vs. t at failure. It is shown that at $t = 0$, F.HC3.500 was in compression due to prestress and there were only some locations where the slab was in tension due to the applied load (Fig. 6.24(a)). However, at failure, due to heating effect, the web of F.HC3.500 were in tension (Fig. 6.24(b)); Failure occurred when the maximum principal tensile stress in the concrete web reached tensile strength of concrete.

Fig. 6.24 Principal-stress comparison: Before heating vs. at failure (F.HC3.500)

6.6 Conclusions

This chapter presents experimental and numerical investigations on the web-shear behavior of deep precast/prestressed concrete hollow core (PCHC) slabs subjected to fire. Based on the results and observations presented and discussed above, the following findings can be drawn:

1. Four types of crack were sequentially observed in the structural fire shear tests, i.e. splitting, longitudinal, thermal, and diagonal web-shear cracks. The formation of web-shear cracks substantially reduced the load-carrying capacity of the slabs, directly causing brittle failure.
2. The proposed FE models were capable of tracing temperature development and predicting web-shear behavior of PCHC slabs under fire conditions. In addition, thermal, longitudinal and web-shear cracks were well predicted by the FE models. However, the models assumed perfect bonding between the concrete and the strand. Therefore, splitting cracks and anchorage failure could not be simulated. This is a limitation of the developed FE models.
3. When subjected to the ISO-834 fire curve, numerical investigations showed that F.HC1.320 failed at 83 min of fire exposure while the other three slabs with depth of 400 mm and 500 mm failed within 60 min of fire exposure, which represented premature failure. This finding agreed well with experimental results from Fellingner (2004) on 400 mm-thick hollow-core slabs exposed to the standard fire.
4. With the same load level of 40%, a clear trend of deeper slabs having lower web-shear resistance was observed. Besides, it is evident that temperature-induced tensile stresses, instead of temperature-induced degradation in strength, governed the web-shear behavior of deep PCHC slabs.
5. Load level has a significant effect on the web-shear behavior of PCHC slabs under fire conditions. Numerical studies showed that, while the 500 mm thick specimen with a load level of 40% failed at 24 min of fire exposure, it could achieve 106- and 159-min of fire ratings, if the load level was decreased to 20% and 10%, respectively.
6. Finally, the prescriptive method in Eurocode 2 which is based on the axis distance and thickness of concrete members provides unconservative design for fire resistance of deep PCHC slabs subjected to shear. In contrast, the formula in EN

1168 gives safe predictions for shear strength of PCHC slabs under elevated temperatures.

Experimental and numerical studies on conventional PCHC slabs presented in this chapter together with those in Chapter 4 showed that web-shear failure is critical for hollow-core slabs at both ambient and fire conditions. The next chapter (Chapter 7) investigates the use of fibers including PP and steel fibers with different contents as a shear-strengthening technique to enhance shear capacity of PCHC slabs.

CHAPTER 7 EFFECT OF POLYPROPYLENE AND STEEL FIBERS ON WEB-SHEAR RESISTANCE OF PCHC SLABS (SERIES III)

7.1 Introduction

Given that stirrups cannot be implemented in PCHC slabs and web-shear failure is critical as shown in Chapter 4 and Chapter 6, this chapter investigates the use of polypropylene (PP) and steel fibers to improve shear strength and ductility of hollow-core slabs. Twelve shear tests conducted on six hollow-core specimens with and without fibers (Series III) are presented in this chapter. The investigations aimed to (1) quantify the effectiveness of incorporating PP, hooked and high-strength/straight steel fibers with different contents on shear strength of PCHC slabs; (2) investigate the potential use of such fibers in extruded hollow-core slabs in practice; (3) develop a semi-empirical formula to predict shear strength of hollow-core slabs with steel fibers. Experimental and code predictions of the twelve test results (presented in this chapter) and 39 tests collected from the literature, coupled with actual implementation of the three types of fiber in a precast plant, provide a comprehensive study on enhanced shear capacity of extruded hollow-core slabs by fibers.

The overview of Series III of the test program was introduced in Section 3.3.1. Geometric and material properties of the six specimens were also given in Section 3.3.2. This chapter presents detailed test setup, instrumentation, test results and discussions of Series III. Additionally, code comparisons are also discussed, followed by a semi-empirical formula proposed to predict shear strength of SFRC hollow-core slabs.

7.2 Test setup and instrumentation of Series III

In this series, twelve full-scale shear tests were conducted on six hollow-core slabs (with and without fibers) to quantify the effectiveness of using different types of fiber in PCHC

slabs. Three types of fiber with different fiber-volume fractions, i.e. PP fibers with fiber contents of 0.11 and 0.22%, hooked steel fibers with two volume fractions of 0.51 and 0.89%, and high-strength/straight steel fibers with a content of 0.89%, were examined. The test program is summarized in Table 7.1.

Table 7.1 Summary of test program

Slab ID.	Test ID.	Span length, L^* (m)	Effective depth, d (mm)	Cantilever length, L_c^* (mm)	Fiber type	Fiber content in kg/m ³ (fiber- volume fraction, %)
HC1.0	HC1.0.a	4.00	355	75	n.a	n.a
	HC1.0.b	2.95	355	1125	n.a	n.a
HC2.PP.1	HC2.PP.1.a	4.00	356	75	PP fiber	1 (0.11)
	HC2.PP.1.b	2.95	356	1125	PP fiber	1 (0.11)
HC3.PP.2	HC3.PP.2.a	4.00	358	75	PP fiber	2 (0.22)
	HC3.PP.2.b	2.95	358	1125	PP fiber	2 (0.22)
HC4.H.40	HC4.H.40.a	4.00	365	75	H fiber	40 (0.51)
	HC4.H.40.b	2.95	365	1125	H fiber	40 (0.51)
HC5.H.70	HC5.H.70.a	4.00	365	75	H fiber	70 (0.89)
	HC5.H.70.b	2.95	365	1125	H fiber	70 (0.89)
HC6.S.70	HC6.S.70.a	4.00	365	75	S fiber	70 (0.89)
	HC6.S.70.b	2.55	365	1525	S fiber	70 (0.89)

*Refer to Fig. 7.1

The setup of the twelve tests is shown in Fig. 7.1 and was in accordance with the shear-test arrangement on hollow-core slabs specified in EN 1168 (EN-1168:2005+A3 2011). In all the tests, the specimens were simply supported and loaded by an unsymmetrical line load with regard to the center line of the slabs. Two rubber pads of 10 mm thick and 100 mm wide were placed between the slabs and the two steel bearings (at support locations), and a cement bed with the same dimension was applied between the specimen and the steel roller at the applied load location (Fig. 7.1). These were adopted to compensate for any unevenness of the slab surface and allow a uniform stress distribution at the contact locations. Shear span, following EN 1168 (EN-1168:2005+A3 2011), was taken as $2.5h$ (in which h is the overall slab depth), resulting in a shear span of 1000 mm in all the tests. The monotonically increasing load was applied using a 200-ton hydraulic jack and monitored by a load cell placed between the jack and the spreader beam. Vertical displacements along the slab length were measured using eight linear variable displacement transducers (LVDTs) (Fig. 7.1). The twelve specimens were loaded till failure. Crack propagation, cracking and ultimate loads, as well as failure modes were also monitored during the tests.

Fig. 7.1 Test configuration

7.3 Test results and discussions

7.3.1 Test results

Test results from the twelve tests are summarized in Table 7.2. It should be noted that ultimate shear strength, V_u , as reported in Table 7.2, was calculated from the peak load (P_u) using the formula in Fig. 7.2. In addition, the deflections shown in Table 7.2 were the ultimate deflections (under the loading point) at which the specimens were unable to resist the imposed load P from the jack. As a result, load resistance dropped significantly.

Fig. 7.2 Shear-force calculation

Table 7.2 Test results

Test ID.	Cracking load (kN)	Peak load, P_u (kN)	Ultimate shear strength, V_u (kN)	Ultimate shear stress $(V_u/b_w d)^*$ (MPa)	Ultimate deflection (mm)	Failure mode**
HC1.0.a	358	358	269	2.76	1.55	WS
HC1.0.b	421	421	278	2.85	1.01	WS
HC2.PP.1.a	487	487	365	3.73	2.21	WS
HC2.PP.1.b	491	491	325	3.32	2.12	WS
HC3.PP.2.a	452	452	339	3.44	2.20	WS & A
HC3.PP.2.b	404	407	269	2.73	1.71	WS & A
HC4.H.40.a	496	496	372	3.71	18.15	WS
HC4.H.40.b	613	615	406	4.04	17.44	WS
HC5.H.70.a	734	741	556	5.54	20.03	FS
HC5.H.70.b	672	672	444	4.42	21.51	WS
HC6.S.70.a	870	923	693	6.90	28.53	FS
HC6.S.70.b	990	994	604	6.02	18.34	FS

* b_w = total web width; d = effective depth of slabs;

**WS = Web-shear; FS = Flexural-shear; A = Anchorage failure; (Fig. 7.4)

7.3.2 Overall behavior of test slabs

Load-displacement responses of the slabs are shown in Fig. 7.3. The curves of specimens with comparable span lengths are plotted in a single figure for comparison purpose. As can be seen from Fig. 7.3, with the same shear span of 1000 mm corresponding to a shear-span-to-effective-depth ratio ranging from 2.74 to 2.82, behavior of the specimens can be divided into two groups, namely, Group A and Group B. While Group A includes the six specimens without steel fibers (HC1.0.a&b, HC2.PP.1.a&b, and HC3.PP.2.a&b), Group B consists of the other six specimens with hooked or straight steel fibers (HC4.H.40.a&b, HC5.H.70.a&b, and HC6.S.70.a&b). Fig. 7.3 shows that the specimens in Group A exhibited brittle behavior, whereas the slabs in Group B failed in a much

more ductile manner. This indicated that the use of PP fibers in PCHC slabs up to 0.22 % by volume did not improve post-cracking behavior. Brittle failure still occurred as it did in the companion hollow-core slab without fibers (HC1.0).

a) Specimens with a span length of 4.0 m

b) Specimens with span lengths of 2.95 and 2.55 m

Fig. 7.3 Load-displacement curves

On the other hand, when steel fibers were adopted, the specimens in Group B behaved in a much more ductile manner. Although cracking loads of these specimens were almost similar with ultimate shear force (except HC6.S.70.a which could resist an additional 5.7% of peak load after cracking (Table 7.2)), all the slabs in Group B failed gradually

at much larger deformations. Thus, slabs with steel fibers could absorb a greater amount of energy compared to those with PP fibers or without any fibers. Clearly, post-cracking behavior of hollow-core units with steel fibers was significantly improved.

7.3.3 Failure modes

Failure modes of specimens are listed in Table 7.2 and typically shown in Fig. 7.4. Three types of failure were observed: (1) web-shear (WS); (2) a combination of web-shear and anchorage (WS & A); and (3) a combination of flexural and shear (FS), the so-called flexural-shear failure. In the first mode of failure, cracks initiated in concrete webs near the support and immediately propagated towards the bearing and loading locations, resulting in a sudden failure without warning with the formation of diagonal web-shear cracks as shown in Fig. 7.4(a). In the second mode of failure, web-shear failure was combined with anchorage failure and the web-shear cracks occurred almost concurrently with cracks along longitudinal strands (Fig. 7.4(b)). Loss of bond between the concrete and the prestressing strands due to splitting resulted in longitudinal cracks at the slab edge regions. In the last mode of failure, flexural cracks first occurred at the bottom of the slabs and then propagated diagonally towards the applied-load location, forming shear cracks as illustrated in Fig. 7.4(c). The slabs failed due to interaction between flexure and shear modes.

As shown in Table 7.2, all the specimens without steel fibers failed in web-shear or a combination of web-shear and anchorage modes. However, when steel fibers were included, in some instances, failure mode changed from web-shear to flexural-shear failure, as shown in HC5.H.70.a and HC6.S.70.a&b. This was because when the tensile strength of concrete in the webs was considerably strengthened by steel fibers, web-shear cracks could not emanate; instead, flexural cracks at the bottom of slabs occurred first and then propagated vertically and diagonally towards the applied load location, resulting in flexural-shear failure. For slabs that failed in flexural-shear mode, shear strengths calculated from the applied loads (Table 7.2) were not equal to their web-shear strength; instead, they were just lower bounds of the web-shear strength.

Fig. 7.4 Failure modes in test slabs

7.3.4 The effect of fibers on shear strength of test specimens

The effectiveness of PP and steel fibers on ultimate shear strength of hollow-core slabs is summarized in Table 7.3. In general, adding fibers brought positive effect on ultimate shear strength of PCHC slabs compared to the control specimens (HC1.0.a&b) without any fibers.

Table 7.3 Increase in shear strength

Slab ID.	Fiber type	Fiber- volume fraction (%)	Average shear strength (kN) / <i>COV</i> * (%)	Average ultimate shear stress ($V_u/b_w d$) (MPa)	Increase in ultimate shear strength (%)
HC1.0	-	0	267 / 2.3	2.8	n.a
HC2.PP.1	PP fiber	0.11	345 / 8.2	3.5	29.2
HC3.PP.2	PP fiber	0.22	304 / 16.3	3.1	13.9
HC4.H.40	H fiber	0.51	389 / 6.2	4.0	45.8
HC5.H.70	H fiber	0.89	500 / 15.9	5.0	98.6
HC6.S.70	S fiber	0.89	649 / 9.7	6.5	142.9

* *COV* = coefficient of variation***Discussion on the effect of PP fibers on shear strength***

From Table 7.3, with the use of 0.11 and 0.22% PP fibers, the respective shear strength of the slabs increased by 29.2 and 13.9% compared to that of HC1.0 (the control specimen without fibers). The increase in ultimate shear strength of HC2.PP.1 and HC3.PP.2 was due to an increase in concrete strength in both compression and tension tests due to addition of PP fibers as shown in Table 3.13. However, it should be noted that the shear capacity of HC3.PP.2 was expected to be at least similar to or higher than that of HC2.PP.1. This is because the fiber content in HC3.PP.2 was greater than that of HC2.PP.1 (2 kg/m³ compared to 1 kg/m³). Furthermore, results from the material tests (Table 3.13) showed that the two slabs had comparable concrete strengths in compression and tension. However, results from the structural tests showed the opposite trend. This could possibly be due to non-uniform fiber dispersion in HC2.PP.1 and HC3.PP.2, which is a common issue in concrete members using fibers. In general, adding 0.11 and 0.22% volume fractions of polypropylene fiber increased the web-shear strength of PCHC slabs by between 13.9% and 29.2%.

Discussion on the effect of steel fibers

As can be observed from Table 7.3, shear strength of PCHC slabs significantly increased with steel-fiber content. This is expected since a higher fiber content brings about a higher number of steel fibers to bridge cracks and to contribute to shear resistance. Fig. 7.5 shows distributions of steel fibers over large crack widths of approximately 10 to 15 mm in the three slabs with steel fibers. As can be seen, more fibers were bridging the cracks, and therefore, were mobilized to resist tensile stresses in HC5.H.70 and HC6.S.70 compared to those in HC4.H.40 (Fig. 7.5(b) and (c) vs. Fig. 7.5(a)). This observation agrees well with previous studies on steel-fiber-reinforced concrete beams (Yoon-Keun Kwak and Jubum 2002, Hai H. Dinh and James 2010, Hassan Aoude and Denis 2012) and slabs (Cuenca and Serna 2013, Simasathien and Chao 2015) without stirrups, which showed that hooked steel fibers brought enhancement to shear resistance. However, Simasathien and Chao (2015) pointed out that the use of hooked steel fibers in hollow-core slabs resulted in a greater increase in shear strength compared to beams. This is because the compaction process of extruded hollow-core slabs tends to orientate steel fibers in the vertical direction in the thin webs.

Fig. 7.5 Distribution of steel fibers over cracks

To evaluate the effectiveness of hooked steel fibers versus high-strength/straight steel fibers on shear capacity, test results of HC5.H.70 and HC6.S.70 were compared. With the same volume fraction of 0.89%, ultimate shear stress of HC6.S.70 was 30% greater compared to that of HC5.H.70 (6.5 vs. 5.0 MPa in shown Table 7.3), showing a superior performance in shear resistance when using high-strength/straight fibers. This was not

initially expected since it is generally accepted that slabs with hooked steel fibers would resist higher shear forces when compared to those with straight fibers. This is due to the fact that fiber matrix bond strength mostly comes from the friction between the concrete and the fibers (Dinh 2009). Therefore, hooked steel fibers with deformed ends and higher aspect ratios could produce stronger bonds and promote a higher shear resistance compared to straight fibers. However, it is worth mentioning that the straight fibers used in this study were high-strength fibers with a tensile strength of 2750 MPa, twice that of the hooked steel fibers. In addition, it was observed that concrete workability of HC5.H.70 using hooked steel fibers with a volume fraction of 0.89%, a fiber length of 60 mm and aspect ratio of 80 significantly reduced compared to that of HC6.S.70, of which straight fibers with the same fiber content (0.89%) but a shorter fiber length of 13 mm and a lower aspect ratio of 62 were used. A decrease in workability reduced consistency and compactness of fresh concrete of HC5.H.70. Moreover, due to non-uniform fiber dispersion, congestion of fiber at several locations in HC5.H.70 was observed. At these locations, the hooked steel fibers huddled together and due to aggregate size, concrete mix could not totally fill in the voids inside the fiber clusters. As a result, poor concrete compaction as shown in Fig. 7.6 occurred, reducing shear strength of HC5.H.70. However, the use of straight steel fibers up to 0.89% in volume fraction could increase shear strength by as much as 142.9% compared to that of the companion slab without steel fibers (HC1.0).

Fig. 7.6 Poor concrete compaction at some places in HC5.H.70

7.4 Code predictions of shear strength of test specimens

As shear behavior of the four specimens with PP fibers was similar to that of the control specimens (HC1.0.a&b) as shown in Section 7.3, shear capacity of the six slabs with PP fibers and without any fibers (the control specimens) were computed using ACI 318-14 (ACI 2014) and EN 1168:2005 + A3:2001 (EN-1168:2005+A3 2011), of which the latter is an extension of Eurocode 2 solely for hollow-core slabs. In the calculations, the contribution of PP fibers was taken into account through an increase in concrete compressive and tensile strengths as shown in Table 3.13 (4th and 5th columns). On the other hand, shear strength predictions of the six steel-fiber-reinforced slabs were calculated using the 2010 *fib* model code (FIB 2010), considering the contribution of steel fibers. It is noteworthy that in these calculations, the mean value of concrete strength in Table 3.12 and Table 3.13 instead of characteristic strength was used. In addition, partial safety factors consisting of material safety and strength reduction factors were taken as unity.

7.4.1 Shear strength predictions of slabs without steel fibers

The web-shear strength predictions of the six specimens without steel fibers were calculated using ACI 318-14 and EN 1168. They were compared with the experimental data in Section 7.3 and are shown in Fig. 7.7. As can be seen, the ACI 318-14 formula (with a factor of 0.5 for the “size effect”) provided conservative predictions for the slabs. However, without the size-effect factor, the code overestimated shear strength for four out of the six slabs. Similarly, EN 1168 provided unsafe predictions for shear strength of all the six specimens. This finding agrees well with previous studies (Walraven and Mercx 1983, Brunesi, Bolognini et al. 2015, Nguyen, Tan et al. 2019) on shear behavior of deep hollow-core slabs, which showed that the Eurocode 2 approach is not conservative for web-shear strength predictions of deep PCHC slabs.

Fig. 7.7 Web-shear strength comparisons: experimental results vs. design codes

7.4.2 Shear strength prediction of hollow-core slabs with steel fibers

According to the 2010 *fib* model code (Sec.7.7.3.2.2), shear strength (in N) of steel-fiber-reinforced concrete members without shear reinforcement can be computed as follows:

$$V_{Rd,F} = \left\{ \frac{0.18}{\gamma_c} k \left[100 \rho_1 \left(1 + 7.5 \frac{f_{Ftuk}}{f_{ctk}} \right) f_{ck} \right]^{1/3} + 0.15 \sigma_{cp} \right\} b_w d \quad (\text{Eq.7.7-5})$$

with a minimum value of

$$V_{Rd,F} = (v_{\min} + 0.15 \sigma_{cp}) b_w d \quad (\text{Eq.7.7-6})$$

where

$$v_{\min} = 0.035 k^{3/2} f_{ck}^{1/2}$$

γ_c is a partial safety factor for concrete without fibers;

$k = 1 + \sqrt{\frac{200}{d}}$ is a factor taking into account the size effect;

d is the effective depth (mm);

ρ_l is the longitudinal reinforcement ratio;

f_{Ftk} is the characteristic value of the ultimate residual tensile strength for fiber-reinforced concrete (MPa);

f_{ck} is the characteristic value of cylindrical compressive strength (MPa);

σ_{cp} is the average stress acting on the concrete cross-section due to loading or prestressing actions (MPa);

b_w is the smallest width of the cross section in the tensile area (mm).

Calculated shear strengths of the slabs with steel fibers using the 2010 *fib* model code are compared with experimental results and are shown in Fig. 7.8. The figure indicates that the code provides safe predictions of shear capacity for hollow-core slabs with steel fibers. In addition, the conservatism increases with fiber content. Moreover, with a mean factor of $V_{test}/V_{predict}$ equal to 1.78, the formula exhibits an over-conservatism for shear-strength calculations for hollow-core slabs. This could be due to the fact that the formula was developed and calibrated using steel-fiber-reinforced concrete beams. However, previous studies (Peaston and Paine 1999, Simasathien and Chao 2015, Dudnik, Milliman et al. 2017) have shown that the effectiveness of steel fibers is even more pronounced when they are applied in hollow-core slabs than in beams. The over-conservatism may be due to geometry of hollow-core slabs containing thin webs and compacting process (rotation of augers) of extrusion that tend to orientate steel fibers vertically in the thin webs.

Fig. 7.8 Shear-strength comparison: Experiment vs. the 2010 *fib* model code

7.4.3 Proposed shear strength formula for hollow-core slabs with steel fibers

As the 2010 *fib* model code is overly conservative for shear-strength prediction of hollow-core slabs, a new formula was proposed considering the respective contributions of concrete in compression zone, prestress, steel fibers, and arching action to shear resistance. The formula was then verified using a database of 45 shear tests (6 slabs with steel fibers from this investigation and another 39 steel-fiber-reinforced concrete hollow-core slabs from the literature (Peaston and Paine 1999, Cuenca and Serna 2013, Dudnik, Milliman et al. 2017)).

Proposed formula

It is commonly accepted that shear strength of reinforced concrete beams or slabs without stirrups is provided by longitudinal reinforcement through dowel action (V_d), aggregate interlock (V_a), and the concrete in compression region (V_c). When steel fibers are added, their contributions (V_{fib}) must be included. Total shear resistance of a steel-fiber-reinforced concrete (SFRC) member (V_{SFRC}) as shown in Fig. 7.9(b) can be expressed as follows:

$$V_{SFRC} = V_c + V_a + V_{fib} + V_d \quad (7.1)$$

Fig. 7.9 Shear resistance model of SFRC slabs (partly adapted from Hassan Aoude and Denis (2012))

Of the four components, the contribution of dowel action by longitudinal reinforcement is considered trivial compared to contributions of the other three, especially in concrete slabs with very low reinforcement ratios such as hollow-core slabs (typically, the ratio is within the range of 0.5 to 1.5%). In addition, due to thin webs of PCHC slabs, it is believed that aggregate interlock only has very minor effect on shear resistance. Therefore, in this proposed equation, contributions of dowel action and aggregate interlock are ignored. This assumption was also adopted in previous studies (Bresler and Pister 1958, Tureyen and Frosch 2003, Dinh, Parra-Montesinos et al. 2010) and will result in safe predictions. Thus, the shear resistance of SFRC hollow-core slabs can be further simplified as follows:

$$V_{SFRC} = V_c + V_{fib} \quad (7.2)$$

The contribution of concrete in the presence of prestressing force in the proposed/simplified formula can be computed using Eq. 7.3. In Eq. 7.3, the term of $(f_{spfc} e \frac{d}{a}) b_w d$ accounts for the contribution of concrete to shear resistance, proposed based on a study of shear formulas for reinforced concrete beams which were developed by previous researchers (Zsutty 1971, Yoon-Keun Kwak and Jubum 2002). Their formulas are shown in Appendix D. It should be noted that in the original formulas, some constant values determined by regression analysis were included. However, in the proposed equation, all the constant values were set to unity. On the other hand, the term of $(0.15 \sigma_{cp}) b_w d$ in Eq. 7.3 accounts for the contribution of prestressing steel to shear

strength, taken as recommendation of Eurocode 2 (Eq. 6.2a) (EN-1992-1-1 2004). Eq. 7.3 takes into account combined effects of tensile strength of steel-fiber-reinforced concrete, arching action, and prestressing force on shear resistance.

$$V_c = (f_{spfc} e \frac{d}{a} + 0.15 \sigma_{cp}) b_w d \quad (7.3)$$

where

f_{spfc} is calculated value of split-cylinder strength of fiber concrete using a formula proposed by Narayanan and Darwish (1987):

$$f_{spfc} = f_{cuf} / (20 - \sqrt{F}) + 0.7 + 1.0 \sqrt{F} \text{ (MPa)}$$

$F = (L_f / D_f) V_f d_f$ is fiber factor;

L_f is fiber length (mm);

D_f is fiber diameter (mm);

V_f is volume fraction of steel fiber;

d_f is bond factor (0.5 for round fibers, 0.75 for crimped fibers, and 1.0 for intended and hooked fibers);

$e = 1.0$ if $a/d > 2.8$, and $= 2.8(d/a)$ if $a/d \leq 2.8$ is arch action factor;

σ_{cp} is axial stress at the centroid axis due to effective prestress (MPa);

b_w is the smallest web width (mm);

d is effective depth of slab (mm).

Fiber contribution to shear strength can be calculated using a model partly adapted from Hassan Aoude and Denis (2012) developed for SFRC beams as shown in Fig. 7.9(a). According to Hassan Aoude and Denis, the contribution of steel fibers to shear resistance of hollow-core slabs is based on the force needed to pull out effective fibers. It can be calculated as follows:

$$V_{fib} = F_{pullout} \cos \theta \quad (7.4)$$

where θ is the angle between the crack plane with the longitudinal axis (shown in Fig. 7.9). In this study, θ is taken as 45° as observed in the tests and used in the design codes

(EN-1992-1-1 2004, ACI 2014). The term $F_{pullout}$ is the total pullout force and can be calculated using Eq. 7.5 which is partially adapted from Hassan Aoude and Denis (2012). It should be noted that in the original equation in Hassan Aoude and Denis (2012), the pullout resistance from hooked ends of hooked steel fibers was accounted. However, in this study, it is assumed to be minor and is ignored. The pullout force, therefore, only comes from friction between the steel fibers and the concrete matrix.

$$F_{pullout} = N_{fib} \tau_{bond} \pi D_f \frac{L_f}{2} \quad (7.5)$$

where τ_{bond} is the bond stress between concrete matrix and fibers, taken as 4.15 MPa as recommended by Swamy, Mangat et al. (1974) and N_{fib} is the effective number of steel fibers across the crack plane, which can be computed using Eq. 7.6 as follows:

$$N_{fib} = \left(\frac{V_f}{A_f} \alpha \eta_l \right) \frac{0.9db_w}{\sin \theta} \quad (7.6)$$

In Eq. 7.6, A_f is cross-sectional area of the fiber; α is an orientation factor accounting for the arbitrary orientation of fibers in an cracking plane. It is taken as 3/8 in SFRC beams (Hassan Aoude and Denis 2012). However, as mentioned in Section 7.3.4, due to thin webs and the compacting process of the extrusion method, steel fibers are prone to lie vertically in concrete webs. Therefore, in the proposed equation, the orientation factor is taken as 0.5; η_l is a length factor considering the embedment length of steel fibers when pulled out and is assumed to be 0.5 (Narayanan and Darwish 1987, Hassan Aoude and Denis 2012). The term inside the bracket in Eq. 7.6 stands for the effective number of steel fibers per unit area, following Hannant (1978), while the term outside is the area of the inclined crack plane.

Substituting Eqs. 7.5 and 7.6 into Eq. 7.4, shear resistance due to steel fibers in SFRC hollow-core slabs embodies the form of:

$$V_{fib} = \left(\frac{V_f}{A_f} \alpha \eta_l \right) \frac{0.9db_w}{\sin \theta} \tau_{bond} \pi D_f \frac{L_f}{2} \cos \theta \quad (7.7)$$

Assuming the cross section of steel fibers is round, Eq. 7.7 becomes

$$V_{fib} = (0.45\tau_{bond} \frac{L_f}{D_f} V_f) b_w d \quad (7.8)$$

Substituting Eqs. 7.3 and 7.8 into Eq. 7.2, shear strength of SFRC hollow-core slabs can be calculated as follows:

$$V_{SFRC} = (f_{spfc} e \frac{d}{a} + 0.15\sigma_{cp} + 0.45\tau_{bond} \frac{L_f}{D_f} V_f) b_w d \quad (7.9)$$

As can be seen, Eq. 7.9 accounts for the contribution of steel fibers to the tensile strength of concrete, and therefore to shear strength of PCHC slabs. In addition, it takes into account effects of arching action, prestressing force, fiber content and fiber aspect ratio on shear resistance.

Evaluation of proposed formula (Eq. 7.9)

To verify the proposed model, additional data including shear tests on SFRC hollow-core slabs were compiled from the literature and are summarized in Table 7.4, while detailed information of such tests is given in Table D1 in Appendix D. It should be noted that only tests with a/d ratio ranging from 2 to 5 were included in the database. In addition, only tests with sufficient information to calibrate the expression were selected. Moreover, shear tests carried out on SFRC hollow-core slabs from Simasathien and Chao (2015) were excluded as these hollow-core slabs were cast using conventional normal-slump concrete instead of zero-slump concrete and the extrusion method. Therefore, they may not necessarily reflect actual characteristics of hollow-core slabs in this study. In the end, the database only included a total of 45 slabs with a fiber content varying from 0.5 to 1 percent that failed in shear mode as listed in Table 7.4.

Table 7.4 Shear tests used to verify the proposed formula

Test series	No. of tests	Steel fiber type*	Fiber content (%)	Failure mode**
This investigation	6	H & S	0.51 to 0.89	WS & FS
Peaston and Paine (1999)	12	H	0.5 to 1	WS
Cuenca and Serna (2013)	15	H	0.64 to 0.89	WS & FS
Dud (2015) Dudnik, Milliman et al. (2017)	10	H	0.75	WS
Total	45	H & S	0.5 to 1	WS & FS

*H = Hooked; S = Straight; **WS = Web-shear; FS = Flexural-shear

Fig. 7.10 plots shear-strength results from the tests versus the predictions from Eq. 7.9 while evaluation of the proposed formula is shown in Fig. 7.11. Fig. 7.10 shows that predicted shear strengths correlate well with test data except for the two slabs using high-strength/straight fibers (tested in this study). It should be noted that for the other 43 slabs in the database, hooked steel fibers were used. Thus, it is recommended to limit the use of Eq. 7.9 for shear-strength predictions of hooked-steel-fiber-reinforced concrete hollow-core slabs only. On the other hand, Fig. 7.11 shows that the proposed equation provided a mean of 1.12 for the ratio of experimental shear strength to predicted shear strength with a coefficient of variation (CoV) of 0.21 (Appendix D, Table D1). When the two slabs with high-strength/straight fibers were eliminated from the database, the corresponding mean and CoV were 1.09 and 0.18, respectively. The improvement in shear-strength predictions of the proposed equation was confirmed when the two values are compared to 1.41 and 0.25, i.e. the respective mean and CoV yielded from the *fib* predictions (as shown in Appendix D, Table D2). The verification illustrates that the proposed formula (Eq. 7.9) was capable of estimating shear strength of hooked-steel-fiber-reinforced hollow-core slabs with reasonable accuracy.

Fig. 7.10 Shear strength from experiment vs. predicted shear strength using the proposed equation (Eq. 7.9)

a) V_{exp}/V_{pre} vs. a/d

b) V_{exp}/V_{pre} vs. fiber volume fraction

Fig. 7.11 Evaluation of the proposed formula (Eq. 7.9)

7.5 Conclusions and recommendations

Six hollow-core slabs using different types of fiber and fiber contents were cast using the extrusion method. Twelve shear tests were conducted on the six panels, resulting in two sets of data for each type of fibers used. Based on the experimental and theoretical investigations discussed in this chapter, the following conclusions are drawn:

1. By varying admixture dosage, it was feasible to add polypropylene (PP) (up to 0.22% by volume) and steel fibers (up to 0.89% by volume) in very dry (nearly zero-slump) concrete to be used in the extrusion process. No noticeable problems were encountered during the casting of concrete. However, as hooked steel fibers with a length of 60 mm, an aspect ratio of 80, and a volume fraction of 0.89% were used in HC5.H.70, poor concrete compaction were observed at some locations, causing a reduction in shear resistance. Therefore, it is recommended not to use a combination of 60 mm length hooked steel fibers at a volume fraction of 0.89% in the extrusion method.
2. Shear strength of slabs with PP fiber content of 0.11 and 0.22% by volume increased by as much as 29.2% with a lower bound of 13.9%, compared to the companion slab without fibers. However, post-peak behavior of PP-fiber-

reinforced-concrete slabs was not improved. In addition, brittle failure could not be avoided with the use of PP fibers in hollow-core slabs.

3. Substantial increases in shear strength (up to 142.9%) and toughness were observed when steel fibers with a volume fraction up to 0.89% were used. In addition, shear strength increased with steel fiber content.
4. Web-shear failure was observed in HC4.H.40, of which a lower steel fiber fraction (0.51% by volume) was used. However, as steel fiber content was increased up to 0.89%, in some instances, failure mode changed from web-shear to flexural-shear.
5. As the hollow-core slab with high-strength/straight steel fibers (HC6.S.70) exhibited enhanced shear resistance compared to HC5.H.70 (using hooked steel fibers with the same fiber content), there was a compelling reason to use high-strength/straight steel fibers in hollow-core slabs. Shorter fiber length and finer fiber diameter could help to overcome compaction problems in the extrusion method.
6. It is shown by the authors and other researchers (Peaston and Paine 1999, Simasathien and Chao 2015) that the effectiveness of steel fibers is greater when they are used in hollow-core slabs than in beams. This is due to the thin webs of hollow-core slabs and compacting process of the extrusion method that tend to orientate fibers vertically in the concrete webs. Therefore, fiber content between 0.5 to 0.89% will be sufficient to enhance shear resistance of hollow-core slabs.
7. The *fib* 2010 model code formula for shear strength of SFRC members exhibits an over-conservatism for shear-capacity predictions of hollow-core slabs.
8. A formula to calculate shear capacity of steel-fiber-reinforced concrete hollow-core slabs was proposed and verified. The verification showed that the equation was capable of predicting shear strength of PCHC slabs incorporating steel fibers with reasonable accuracy. However, as the majority of the database (43 out of 45 tests) were conducted on hollow-core slabs using hooked steel fibers, it is

recommended to limit the use of the formula for only hooked-steel-fiber-reinforced concrete hollow-core slabs.

This chapter investigates the use of PP and steel fibers to enhance shear resistance of PCHC slabs at room temperature. The effectiveness of such fibers to improve structural performance of PCHC slabs under fire conditions will be discussed in the following chapter (Chapter 8).

CHAPTER 8 SHEAR BEHAVIOR OF FIBER-REINFORCED CONCRETE HOLLOW-CORE SLABS IN THE EVENT OF FIRE (SERIES IV)

8.1 Introduction

Following the studies on the use of PP and steel fibers to enhance shear performance of PCHC slabs at ambient temperature in Chapter 7, this chapter investigates the effectiveness of using such fibers on PCHC slabs under fire conditions. Full-scale tests on six FRC hollow-core slabs under fire conditions (Series IV) were conducted. FE modeling was subsequently performed, followed by numerical analyses. Experimental and numerical results presented in this chapter helped to extend the understanding behavior of FRC hollow-core slabs under fire conditions and to evaluate the effectiveness of different types of fiber on shear resistance of fire-exposed FRC slabs.

8.2 Experimental program

In this series, full-scale shear tests were conducted on six hollow-core specimens with and without fibers under fire conditions to study the shear behavior of precast/prestressed FRC hollow-core slabs. Three types of fiber with different contents were examined, i.e. PP fibers (volume fractions of 0.11 and 0.22%), hooked steel fibers (0.51 and 0.89% by volume), and high-strength/straight fibers (0.89% by volume). Geometric properties of the six specimens and mechanical material properties of the concrete and the strands used in this series were presented in Chapter 3 (Section 3.3.2). Key information of Series IV is given in Table 8.1.

Table 8.1 Summary of test program

Slab ID.	Fiber type	Fiber content in kg/m ³ (fiber volume fraction, %)	Shear span, a (effective depth, d) (mm)	Failure load at ambient (kN)*	Applied load (kN) (load level (%), % of ambient capacity)
F.HC1.0	n.a	n.a	1000 (355)	365	146 (40)
F.HC2.PP.1	PP fiber	1 (0.11)	1000 (356)	460	184 (40)
F.HC3.PP.2	PP fiber	2 (0.22)	1000 (358)	405	162 (40)
F.HC4.H.40	H fiber	40 (0.51)	1000 (365)	520	208 (40)
F.HC5.H.70	H fiber	70 (0.89)	1000 (365)	668	267 (40)
F.HC6.S.70	S fiber	70 (0.89)	1000 (365)	865	346 (40)

* Defined by tests in Series III

8.2.1 Testing arrangement and instrumentation

The test setup of all the six specimens, in accordance with the shear-test setup for hollow-core slabs specified in EN 1168 (EN-1168:2005+A3 2011), is shown in Fig. 8.1. As can be seen, the specimens were placed on top of an electrical furnace with dimensions of 3 m $\times$ 3 m $\times$ 0.75 m (width $\times$ length $\times$ depth) and simply supported at the two ends by A-frames. As the slab was only 1.2 m wide, open parts of the furnace were covered by steel panels and thermal wool to prevent heat loss (Fig. 8.1(b)). The distance between the center lines of the end support and the line load, the so-called shear span, following EN 1168 (EN-1168:2005+A3 2011), was taken as 1000 mm corresponding to $2.5h$ (h is the overall thickness of the specimens) for all the specimens. In the fire tests, a three-point loading scheme was used, of which the specimens were loaded by an unsymmetrical line load with respect to the center line of the slab as illustrated in Fig. 8.1.

Fig. 8.1 Testing arrangement

In each fire test, one load cell, 12 linear variable displacement transducers (LVDTs) and 10 type-K thermocouples were employed to respectively monitor loads transferred from a 100-ton jack to the specimen, vertical deflections and longitudinal thermal expansions, and temperature development inside the specimen. Load cell and LVDT arrangement are shown in Fig. 8.2 while thermocouple locations are shown in Fig. 8.3. It should be noted that LVDTs 1 to 6 were placed on top of each slab to monitor vertical deflections along the specimen length while longitudinal expansions at the slab's two edges were tracked using LVDTs 7, 9, 10, and 12. Furthermore, LVDTs 8 and 11 were used to measure strand slips during testing. Thermocouples were embedded into the specimens through drilled holes three months before testing. They were installed at 5 or 6 locations over the slab depth at two different sections (Sections A and B) along the specimen length. Detailed locations of thermocouples are shown in Fig. 8.3. After the thermocouples were placed, the drilled holes were subsequently filled using cement paste.

Fig. 8.2 Load cell and LVDT arrangement

Fig. 8.3 Thermocouple arrangement

8.2.2 Testing procedure and heating curves

The fire tests were conducted following a transient state, in which the specimens were initially loaded up to 40% of their ambient capacity defined by the test and subsequently heated up to failure. During heating, the applied loads were maintained constant. Furnace temperature was programed to increase linearly from ambient to 1000 °C at 60 min, resulting in a heating rate of 16 °C/min. Furnace temperature was retained at 1000 °C until the end of the tests. In addition, it is worth mentioning that the heating curve used in this study was non-ISO fire due to limitations of the electrical furnace used. Therefore, results from the tests are used to examine structural behavior of fiber-reinforced-hollow-core slabs in the event of fire instead of determining the fire resistance under the standard

ISO-834 fire. The programmed heating curve together with measured furnace temperatures from the six tests using thermocouples are shown in Fig. 8.4. It is demonstrated that the furnace was well controlled, so that the gas temperature inside the furnace was consistent among all the six tests.

Fig. 8.4 Programmed heating curve vs. measured furnace temperatures

8.3 Test results and discussion

Table 8.2 summarizes test results from all of the specimens. Test results will be discussed in terms of temperature and deflection progression, crack patterns and failure modes. It is worth noting that all the fire tests were terminated once failure occurred. At failure, there were significant drops in load-carrying capacity; as a result, the specimens were unable to withstand the applied loads. Criterion “*R*” specified in EN-1992-1-2 (2004) was violated; exposure time was determined although this value could not be interpreted as “fire resistance”.

Table 8.2 Test results

Test ID.	Shear-span-to-effective-depth ratio (a/d)	Imposed load (kN)	Furnace temperature at failure (°C)	Total exposure time (min)	Mid-span deflection at failure (mm)	Failure mode*
F.HC1.0	1000 (2.82)	146	956	62	24.3	WS
F.HC2.PP.1	1000 (2.80)	184	878	53	19.2	WS
F.HC3.PP.2	1000 (2.79)	162	939	62	18.9	WS
F.HC4.H.40	1000 (2.74)	208	1008	100	58.9	F
F.HC5.H.70	1000 (2.74)	267	1003	88	54.0	F
F.HC6.S.70	1000 (2.74)	346	980	68	50.7	FS

* WS = Web-shear; F = Flexural failure; FS = Flexural-shear

8.3.1 Temperature development

Temperature progression inside each specimen is plotted in Fig. 8.5 where x denotes the distance (mm) from the exposed soffit (bottom slab) to the measured depth. It should be noted that temperature readings from embedded thermocouples at the same depth at Sections A and B (Fig. 8.3) were very consistent. Therefore, only the mean values of the two sets of temperature readings were plotted in Fig. 8.5. As evident, data from thermocouples show reasonable readings since cross-sectional temperatures in concrete over the slab depth decreased as x increased. This is due to low thermal conductivity and high specific heat of concrete that delayed heat transmission over the cross sections (Nguyen and Tan 2018).

a) F.HC1.0

b) F.HC2.PP.1

c) F.HC3.PP.2

d) F.HC4.H.40

e) F.HC5.H.70

f) F.HC6.S.70

Fig. 8.5 Temperature development over specimens' cross section

Fig. 8.5 shows that as fire exposure time increased from 0 to about 42 min, concrete temperatures at measured depths increased gradually from ambient to 100 °C. From 42 min of fire exposure onwards, a temperature plateau at 100 °C was observed. This phenomenon was due to migration of free water content inside the concrete, from layers near the exposed soffit towards the cooler top surface of the slab. As temperature hovered at around 100 °C, water was transformed from liquid state to gas, until the free water in the bottom layers of concrete had vaporized. Moreover, it was also noticed that at several recorded layers where x was equal to or smaller than 120 mm, a sudden increase in concrete temperature occurred in all the specimens (Fig. 8.5). Such an abrupt increase could be due to occurrence of longitudinal cracks at the bottom slab (Fig. 8.10) when gas temperature was about 700°C to 800°C.

8.3.2 Structural responses

The fire tests ended once the slabs showed “runaway” deflections. At these moments, the specimens were unable to sustain the applied loads due to a significant drop in load-bearing capacity. Fig. 8.6 shows the applied load histories together with the development of mid-span deflection.

Fig. 8.6 Applied load and mid-span deflection development vs. time

As can be seen from Fig. 8.6, all the specimens experienced three stages until failure occurred. In the first 25 min of heating (Stage 1), all the slabs deflected downwards at a slow rate due to thermal-bowing effect. Displacements of all the specimens at the end of Stage 1 were about 5 mm at gas temperature of about 430 °C. Stage 2 began with a greater deflection rate due to combined effect of thermal bowing and degradation in strength and stiffness of concrete and strands as temperature further increased. The duration of Stage 2 varied from one slab to another, from 27 min in F.HC2.PP.1 to 74 min in F.HC4.H.40, depending on the type and volume fraction of fibers used (Fig. 8.6). The three slabs with steel fibers could endure longer durations in Stage 2 compared to those with PP fibers and without any fibers. Stage 3 emerged when the furnace

temperature reached at least 850 °C. At the beginning of Stage 3, slab deflection increased at a much faster pace due to a substantial decrease in load-carrying capacity and stiffness of the specimens as a result of elevated temperatures. Stage 3 happened about 3 min, and very soon failure took place.

Fig. 8.7 compares displacement history of the six slabs during fire exposure as measured by LVDT 1 (mid-span deflection). The figure shows that all the three slabs without fibers and with PP fibers failed at gas temperatures lower than 1000 °C and at deflections of about 20 mm. However, displacements at failure of the three slabs with steel fibers were much larger, approximately thrice that of the slabs without steel fibers. Moreover, the three slabs with steel fibers were able to resist the applied loads for much longer durations before failure, showing enhanced fire endurance in terms of strength and ductility. In contrast, structural response of the two slabs with PP fibers was very similar to that of the control specimen without any fibers (F.HC1.0).

Fig. 8.7 Mid-span displacement comparison

8.3.3 Failure mode and crack pattern

Failure mode

Failure modes that occurred in the six specimens are listed in Table 8.2 and are shown in Fig. 8.8. As can be observed, three distinct modes of failure occurred in the tests, i.e. web-shear in F.HC1.0, F.HC2.PP.1 and F.HC3.PP.2, flexural-shear in F.HC6.S.70, and flexural failure in F.HC4.H.40 and F.HC5.H.70.

a) Web shear failure - F.HC1.0

b) Flexural failure – F.HC4.H.40

c) Flexural failure – F.HC5.H.70

d) Flexural-shear failure – F.HC6.S.70

Fig. 8.8 Failure modes of the test specimens

Generally, in web-shear failure, cracks initiated in concrete webs due to tensile stress of concrete in the webs caused by a combination of shear, prestressing force and temperature-induced tensile stress exceeding tensile strength of concrete. Once emanated, the cracks immediately propagated towards the loading and bearing locations, forming diagonal web-shear cracks (Fig. 8.8(a)) and leading to a sudden failure. In flexural-shear failure, flexural cracks first occurred at the bottom of the specimen in a region where bending moment and shear force co-existed (usually between loading and bearing locations). Due to reduced shear resistance in diagonal tension plane as a result of elevated temperatures, the tip of flexural cracks extended diagonally towards the applied load location, forming inclined shear cracks as shown in Fig. 8.8(d). The specimen collapsed due to the combined effect of flexure and shear. In the last failure mode (flexure), cracks initiated at the bottom slab within the loading region. With time, the vertical cracks propagated upwards and widened, exposing prestressing strands to high temperature. Consequently, strand strength degraded significantly, and failure occurred with yielding or rupture of strands as shown in Fig. 8.9(a).

Fig. 8.9 Observations from a flexural crack (F.HC4.H.40): (a) rupture of strands; (b) fibers across the crack

In this study, F.HC1.0, F.HC2.PP.1 and F.HC.PP.2 failed in web-shear mode after sustaining similar durations of fire endurance, demonstrating that the shear resistance of hollow-core slabs under fire conditions was not improved with the inclusion of PP fibers. Brittle behavior occurred in all of the specimens and the slabs suddenly failed without

warning. In contrast, when hooked steel fibers were incorporated, shear failure was completely avoided. F.HC4.H.40 and F.HC5.H.70 (with hooked steel fibers) could sustain applied loads for longer durations, experienced larger deflections before failure, and collapsed in ductile flexural mode. Clearly, fire endurance of hooked-steel-fiber-reinforced hollow-core slabs was significantly enhanced. Fire behavior of F.HC6.S.70 was in between the slabs with hooked steel fibers (F.HC4.H.40 & F.HC5.H.70) and the slabs without steel fibers (F.HC1.0, F.HC2.PP.1, and F.HC3.PP.2); F.HC6.S.70 with high-strength/straight fibers failed due to an interaction between flexural and shear (flexural-shear failure) when furnace temperature reached 1000 °C, suggesting a moderate enhancement in shear resistance.

Development of crack patterns in the test specimens

During the fire tests, five types of cracks were sequentially observed, i.e. splitting, longitudinal, thermal, web-shear, and flexural cracks. It is noteworthy that not all the mentioned cracks occurred in one test. Table 8.3 lists the types of crack observed in each of the six tests while these crack patterns are shown in Fig. 8.10 to Fig. 8.12. In addition, during heating, temperature-induced spalling was observed in all the slabs with steel fibers and without any fibers. However, no spalling was found in any of the two specimens with PP fibers, showing the effectiveness of PP fibers in enhancing spalling resistance. Mechanisms of which PP fibers contribute to prevent spalling of concrete will be discussed in greater detail in Section 8.5.1.

Table 8.3 Types of crack occurred in the test

Slab ID.	(1)* Splitting cracks	(2)* Longitudinal cracks	(3)* Thermal cracks	(4)* Web- shear cracks	(5)* Flexural cracks	Spalling	Failure mode**
F.HC1.0	√	√	√	√		√	WS
F.HC2.PP.1	√	√	√	√			WS
F.HC3.PP.2	√	√	√	√			WS
F.HC4.H.40			√		√	√	F
F.HC5.H.70			√		√	√	F
F.HC6.S.70			√	√	√	√	FS

*With the exception of F.HC6.S.70 (both web-shear and flexural cracks were observed at failure), the sequence of formation: (1) → (5); ** WS = Web-shear failure; F = Flexural failure; FS = Flexural-shear failure;

Fig. 8.10 Splitting and longitudinal cracks

Fig. 8.11 Thermal cracks occurred in all the test specimens due to temperature-induced stress

Fig. 8.12 Web-shear and flexural cracks (causing failure)

Splitting cracks between the concrete and the strands as shown in Fig. 8.10(a) first emerged at the two slab supporting edges, at gas temperature of about 700°C, as a result of incompatible thermal expansion between the concrete and the strands. Incompatibility in thermal expansion between the concrete and the strand induced additional longitudinal shear stresses between them, promoting localized splitting cracks to occur. Longitudinal cracks (Fig. 8.10(b)), on the other hand, occurred at the top and bottom surfaces of hollow cores due to thermal expansion of concrete in transverse direction. These cracks started at the two slab supporting edges. Due to an absence of steel meshes at the top and bottom surfaces of the slab, these cracks propagated towards the slab center, forming longitudinal cracks. It is shown that while longitudinal cracks developed from one end

to another end in three slabs without fibers and with PP fibers (F.HC1.0, F.HC2.PP.1, and F.HC3.PP.2), there was no such cracks observed in the three slabs with steel fibers (F.HC4.H.40, F.HC5.H.70 and F.HC6.S.70) (Fig. 8.13). This is because tensile strength of concrete was significantly increased with inclusion of steel fibers as shown in Table 3.13. Moreover, when steel fibers were added, they were beneficial to the slab by mobilizing tensile stresses, thus delaying the growth of cracks.

a) Splitting and longitudinal cracks occurred in F.HC1.0, F.HC2.PP.1, and F.HC3.PP.2

b) No splitting and longitudinal cracks occurred in F.HC4.H.40, F.HC5.H.70, and F.HC6.S.70

Fig. 8.13 Splitting and longitudinal cracks in the test specimens

In contrast, vertical thermal cracks (Fig. 8.11(a)) occurred in all the slabs due to non-linear temperature gradient over the depth of the specimen, inducing thermal stresses over the cross section. Previous studies (Borgogno and Fontana 2000, Fellingner 2004) have shown that temperature gradient in hollow-core slabs, of which concrete was hotter in the bottom of the slab and cooler at the top, resulted in tensile stress in the web regions as shown in Fig. 8.11(b). This temperature-induced thermal stresses promoted vertical thermal cracks to occur along the concrete webs. The fire tests showed that the formation of splitting, longitudinal and thermal cracks did not immediately lead to failure. However, the formation of web-shear cracks (Fig. 8.12(a)) or the opening of flexural cracks (Fig. 8.12(b)) caused a significant decrease in load-carrying capacity, triggering failure to occur.

8.4 Numerical investigations

In this section, finite element (FE) modeling and numerical analysis using ABAQUS (Simulia 2014) are presented. Two FE models aimed to simulate thermal and structural responses of PCHC slabs with and without fibers in the event of fire were developed. They were verified using test data from the experimental program presented in Section 8.3. The verified FE models were then used to further investigate web-shear behavior of PCHC slabs under fire conditions.

Modeling technique to simulate PCHC slabs under elevated temperatures was introduced in Chapter 6 (Sections 6.4.1 and 6.4.2). This section emphasizes on constitutive model for FRC and numerical results.

8.4.1 Constitutive model

Eurocode 2 constitutive model for prestressing strands (EN-1992-1-2 2004) was selected to simulate temperature-dependent stress-strain behavior of the strands while the *Concrete Damage Plasticity model* (the CDP model) specified Abaqus/explicit (Simulia 2014) was adopted to simulate temperature-dependent nonlinear behavior of concrete.

In the CDP model, concrete behavior under uniaxial stress state in compression and tension must be defined. The temperature-dependent stress-strain curves in compression for all the six slabs were developed complying with Eurocode 2 Part 1-2 (EN-1992-1-2 2004) for plain concrete. This is because the model code *fib* 2010 (FIB 2010) suggests that stress-strain curves for plain concrete can be applied to fiber-reinforced concrete. However, tension behavior of plain concrete and steel-fiber-reinforced concrete must be considered separately as it is shown in *fib* 2010 (FIB 2010) that there is a huge difference in post-cracking relations between plain concrete and steel-fiber-reinforced concrete (Fig. 8.14).

Fig. 8.14 Behavior of SFRC concrete in flexural tensile strength tests (adapted from *fib* 2010 (FIB 2010))

The experimental studies presented in Section 8.3 showed that fire behavior of the two slabs with PP fibers (F.HC2.PP.1 and F.HC3.PP.2) was similar to that of the control slab without any fibers (F.HC1.0). Post-cracking behavior of F.HC2.PP.1 and F.HC3.PP.2 was not improved, and brittle failure occurred even with the use of PP fibers. Therefore, in this numerical analysis, the stress-crack opening relation proposed by Hordijk (1992) for plain concrete in tension was adopted in all the three slabs without steel fibers (F.HC1.0, F.HC2.PP.1 and F.HC3.PP.2). On the other hand, the post-cracking relation in tension proposed by *fib* 2010 for SFRC was used for the three slabs with steel fibers (F.HC4.H.40, F.HC5.H.70, and F.HC6.S.70). Residual flexural tensile strengths of the three slabs defined by material tests as shown in Table 3.12 were incorporated into the analyses. With regard to temperature-induced degradation in strength, concrete tensile strength reduction factors specified in Eurocode 2 Part 1-2 (EN-1992-1-2 2004) were adopted in both concrete with and without steel fibers. Herein, it was assumed that the strength reduction factors for plain concrete and SFRC were the same.

8.4.2 Model verification

The FE heat transfer model

As concrete thermal and section properties of the six specimens in the experimental program were identical, only one FE heat transfer model was built. Temperature development inside the modeled hollow-core slab from the FE heat transfer model was then applied to all the six FE mechanical models for structural analyses. Fig. 8.15 shows

the predicted temperature distribution over the cross section of the PCHC slabs at different times. The numerical heat transfer model was verified by comparing predicted temperatures (FEM) with recorded temperatures from experiments. Fig. 8.16 representatively shows temperature comparisons between the experiments and the numerical simulations at two slab depths ($x = 50$ mm and $x = 90$ mm). It is shown that the predicted temperatures correlated well with the test data, thus verifying the FE heat transfer model. Therefore, temperature results from the heat transfer model were reliable and could subsequently be applied to the FE mechanical model for the analysis of PCHC slabs under fire conditions.

Fig. 8.15 Temperature distribution of test specimen at different times – FE results

Fig. 8.16 Temperature comparisons between the test and the FE heat transfer model

The FE mechanical model

The developed FE mechanical models were verified using the experimental results from the six fire tests. Deflection development, failure mode, and furnace temperature at failure were compared. It should be noted that in the FE models, the six slabs failed when there were significant drops in load-bearing capacity due to formation of web-shear or flexural-shear cracks in shear failure mode or rupture of strands in flexural failure mode. At these moments, the models stopped running due to excessive distortions of concrete elements. Fig. 8.17 shows comparisons between the tests and the FE models for the three slabs failed in web-shear mode (F.HC1.0, F.HC2.PP.1, and F.HC3.PP.2). As can be seen, reasonable correlation in terms of displacement development, maximum displacement and furnace temperature at failure was obtained between the tests and the FEM. Moreover, web-shear failure mode was observed for all the three slabs in the FEM (representatively shown in Fig. 8.17(b). It should be noted that concrete damages in the FE models were represented through plastic strain (PE). The FEM shows that brittle failure still occurred and post-cracking behavior was not improved in the two slabs with PP fibers. This result agrees well with the test data shown in Section 8.3. Furthermore, thermal and longitudinal cracks in the tests were well captured in the FE models as shown in Fig. 8.18. However, it should be noted that splitting cracks between the concrete and the strands occurred in the tests without steel fibers, but this behavior could not be simulated in the FE models due to the perfect bond assumption between the steel strand and the concrete. Most importantly, the comparisons in terms of displacement development, furnace temperature at failure, and failure mode show that the developed FE mechanical models were capable of simulating web-shear behavior of PCHC slabs with and without PP fibers under fire conditions.

Fig. 8.17 Comparison between Tests vs. FEM

Fig. 8.18 Comparisons of crack patterns between the test vs. FE model (F.HC1.0)

Comparisons between the tests and the FEM for the three slabs that failed in flexural-shear mode (F.HC6.S.70) and flexural mode (F.HC4.H.40 and F.HC5.H.70) are shown in Fig. 8.19 to Fig. 8.21. Deflection developments, furnace temperatures at failure, and failure modes were compared. The three figures show that flexural-shear failure in F.HC6.S.70 (Fig. 8.21(b)) and flexural failure in F.HC4.H.40 (Fig. 8.19(b)) and F.HC5.H.70 (Fig. 8.20(b)) were closely simulated. Furnace temperatures at failure of all the three slabs were also reasonably predicted in the FE models. However, there were discrepancies in displacement-time curves between the tests and the FE models (Fig. 8.19(a), Fig. 8.20(a), Fig. 8.21(a)). The three slabs in the FE models experienced slower deflection rates after the furnace had reached the maximum temperature of 1000°C (after 60 min of fire exposure). They were able to resist longer fire-endurance times compared to the tested slabs. It is believed that in the tests, fire-induced spalling at the slab bottom occurred when the furnace temperature was about 670°C to 700°C , causing the strands

to expose to higher temperatures. Consequently, the strength and stiffness of the strands and the concrete at the bottom parts degraded quickly, causing greater rates of deflection. Since the FE models could not simulate spalling in the bottom slab, the simulated slabs lasted longer than the tested slabs (Fig. 8.19(a) to Fig. 8.21(a)).

To confirm the hypothesis, spalling effect was manually incorporated in the FE models by applying higher temperatures to the strands. The axis distance from the fire exposed soffit to the strands was 35 mm. After furnace temperature reached 700°C and spalling occurred as observed in the tests, temperature profiles applied to the strands in the FEM were increased. As spalling depth was not measured in the tests, two values of 10 mm and 15 mm for the depth of spalling were assumed. Strand temperatures were increased to concrete temperatures at the level of 25 mm (if spalling depth of 10 mm was assumed) and 20 mm (if spalling depth of 15 mm was assumed) from the bottom soffit. Fig. 8.19(a) shows that when spalling effect was manually incorporated, better agreement in deflections between the tests and the FEM was obtained. It is also shown that F.HC4.H.40 in the FE model failed earlier when spalling depth increased (Fig. 8.19(a)). Similarly, when spalling effect was incorporated, better correlations in displacement development between the tests and the FEM were obtained for F.HC5.H.70 and F.HC6.S.70 as shown in Fig. 8.20(a) and Fig. 8.21(a), respectively.

a) Displacement-development comparison

b) Failure-mode comparison

Fig. 8.19 Comparison between the test vs. FE model - F.HC4.H.40

a) Displacement-development comparison

b) Failure-mode comparison

Fig. 8.20 Comparison between the test vs. FE model - F.HC5.H.70

Fig. 8.21 Comparison between the test vs. FE model - F.HC6.S.70

The comparison between the tests and the FE models for all the six slabs are summarized in Table 8.4. It is shown that even when spalling was not simulated in the numerical models, good agreement was obtained between the tests and the numerical predictions in terms of furnace temperature at failure, maximum deflection and failure mode (important indicators for anticipation of failure). The verification showed that the developed FE models could predict behavior of PCHC slabs with and without fibers under fire conditions.

Table 8.4 Comparison between the tests and the developed FE models (without considering spalling)

	Furnace tempt at failure (°C)		Mid-span deflection at failure (mm)		Failure mode*	
	Test (FEM)	Test/FEM	Test (FEM)	Test/FEM	Test	FEM
F.HC1.0	965 (902)	1.07	24.2 (23.8)	1.02	WS	WS
F.HC2.PP.1	878 (863)	1.02	19.2 (22.5)	0.85	WS	WS
F.HC3.PP.2	939 (896)	1.05	18.9 (19.8)	0.95	WS	WS
F.HC4.H.40	1008 (1009)	1.00	58.9 (53.8)	1.09	F	F
F.HC5.H.70	1003 (1003)	1.00	54.0 (53.2)	1.02	F	F
F.HC6.S.70	980 (1008)	0.97	50.7 (51.5)	0.98	FS	FS

* WS = Web-shear failure; F = Flexural failure; FS = Flexural-shear failure;

8.4.3 Temperature-induced tensile stress – Numerical analysis

It is shown in both the experiments and the FE models that web-shear failure is sudden and brittle. PCHC slabs failed in web-shear mode before furnace temperature reached 1000 °C, showing premature failure. The verified FE mechanical models were employed to investigate the effect of temperature to web-shear behavior of PCHC slabs. F.HC2.PP.1 was randomly selected for this investigation.

Theoretically, web-shear failure occurred when the maximum principal tensile stress in the concrete web reached tensile strength of concrete. Fig. 8.22 compares principal-stress distribution inside F.HC2.PP.1 at two different times: $t = 0$ (after loading and before heating) vs. t at failure. As shown in the figure, before heating, F.HC2.PP.1 was in compression due to prestress. There were several locations where the slab was in tension due to loading effect (Fig. 8.22(a)). However, when heat was applied, the entire web was in tension at the time of failure (Fig. 8.22(b)). The FE results agreed well with theoretical results from previous studies (Borgogno and Fontana 2000, Fellingner 2004, Nguyen 2014), which showed that nonlinear thermal gradient across the section produced tensile stresses in concrete webs.

Fig. 8.22 Principal-stress comparison: Before heating vs. at failure (F.HC2.PP.1)

Normal stresses of F.HC2.PP.1 in three directions (transverse (S11), vertical (S22), and longitudinal (S33)) at $t = 0$ and t at failure were compared in Fig. 8.23, Fig. 8.24, and Fig. 8.25, respectively. It is shown that under the heating effect, the concrete webs were subjected to tensile stresses in all the three directions, making them much more vulnerable under temperature-load effect.

Fig. 8.23 Normal stress in transverse direction: Before heating vs. at failure (F.HC2.PP.1)

Fig. 8.24 Normal stress in vertical direction: Before heating vs. at failure (F.HC2.PP.1)

Fig. 8.25 Normal stress in longitudinal direction: Before heating vs. at failure (F.HC2.PP.1)

Temperature profiles from the test and the FEM show that at failure, temperature of strands inside F.HC2.PP.1 had not reached 200 °C (Fig. 8.16(a)) and temperature of mid-web concrete was only about 70 °C (Fig. 8.5(b)). At such temperature, based on Eurocode 2 (EN-1992-1-2 2004), residual strength of the strands were about 90% and the concrete strength was about the same with that at ambient. Therefore, results from this numerical analysis agreed with the findings from Chapter 6 that temperature-induced tensile stresses instead of temperature-induced reduction in strength of concrete and

prestressing steels played a decisive role and were mainly responsible for web-shear failure of PCHC slabs. Additionally, stress fields from the numerical analysis also helped to explain how web-shear failure could be avoided in the three SFRC slabs. Basically, when steel fibers were added, tensile strength of concrete was considerably enhanced. Moreover, with random distribution, the fibers were able to resist tensile stresses in different directions. As a result, web-shear cracks could not be mitigated. The SFRC slabs would fail in other failure modes (flexural-shear or flexure) which were triggered from the slab bottom due to degradation in strength of the concrete and the pretensioned strands due to elevated temperatures.

8.5 Discussion on the effect of fibers on fire behavior of PCHC slabs

8.5.1 The effect of PP fibers

Both experimental and numerical results presented in Sections 8.3 and 8.4 showed that the two specimens with PP fiber contents of 0.11 and 0.22% behaved in a similar way compared to the control specimen without fibers (F.HC1.0) in terms of deflection progression, crack-pattern development, failure mode and failure time, indicating that PP fibers were not beneficial for improving load-carrying capacity of PCHC slabs exposed to fire. This is because microcracks (discussed in detail in the next two paragraphs) which occurred at early stage in concrete matrix due to incompatible thermal expansion between the concrete and PP fibers significantly reduced the strength of concrete, resulting in low resistance to applied load of PCHC slabs under fire conditions. In addition, polymer fibers started to melt when subjected to temperature of about 170 °C (Gallé and Sercombe 2001, Khoury and Willoughby 2008, Zhang, Dasari et al. 2018). Beyond this point, PP fibers eventually disappeared, no longer contributing to structural performance of concrete members.

In another aspect, results from the experiments showed that PP fibers played an important role in prevention of concrete spalling. The two slabs with PP fibers exhibited good integrity in terms of spalling prevention while all the other four slabs experienced severe

spalling when exposed to furnace temperature of about 650 °C. Images from scanning electron microscope (SEM) were used to identify changes in concrete with PP fibers before heating and after exposure to 200 °C. Microcracks observed in the concrete (at micro-level) can be used to illustrate how PP fibers prevent spalling of concrete. Details of sample preparation and experimental setup for obtaining SEM images can be found in Zhang, Dasari et al. (2018) and Li, Tan et al. (2019). Microstructural observation in a concrete sample of F.HC3.PP.2 before heating and after heating up to 200 °C is shown in Fig. 8.26.

Fig. 8.26 Microstructural observation in F.HC3.PP.2 sample using SEM images

As can be seen from Fig. 8.26, PP fibers disappeared and microcracks formed in the radial direction of fiber tunnels after the concrete sample was heated up to 200 °C. The microcracks occurred due to a huge difference in the coefficient of thermal expansion between PP fibers and the concrete matrix (Khoury and Willoughby 2008, Zhang, Dasari et al. 2018). Zhang, Dasari et al. (2018) pointed out that at 105 °C, coefficient of thermal expansion of PP fibers was about 41 times greater than that of the concrete matrix. The huge thermal-expansion incompatibility between the two caused tensile stresses in the tangential direction along the fiber-matrix interface, forming an inter-connected fiber-tunnel-microcrack network, which provided pathways for water to escape. According to Zhang, Dasari et al. (2018), the occurrence of microcracks prior to melting of PP fibers played a decisive role in preventing spalling of concrete. In general, experimental results from this study showed that with a PP fiber content of only 1 kg/m³ (0.11% by volume),

spalling of concrete could be prevented. Thus, the use of up to 2 kg/m^3 of PP fibers may not be necessary for spalling prevention in PCHC slabs with a target mean strength of 48 MPa.

8.5.2 Steel fibers enhancing behavior of PCHC slabs exposed to fire

It is shown in the experimental and numerical investigations that the three slabs with steel fibers (F.HC4.H.40, F.HC5.H.70, F.HC6.S.70) could support higher imposed loads and sustain longer heating duration, displaying larger deflections before failure compared to the control specimen without any fibers (F.HC1.0). In addition, while splitting and longitudinal cracks were generated in all the specimens without fibers and with PP fibers, no such cracks occurred in any of the three slabs with steel fibers. Therefore, integrity of the slabs with steel fibers subjected to fire was vastly improved. Furthermore, with the use of steel fibers, web-shear failure mode was completely avoided. The experimental and numerical results demonstrate that shear capacity and fire endurance of hollow-core slabs using steel fibers were significantly enhanced.

To evaluate the use of different steel fiber contents on shear resistance, structural behavior of F.HC4.H.40 and F.HC5.H.70 was compared. It is shown that the two slabs responded to load and temperature in a similar way in terms of deflection (Fig. 8.6(d) vs. Fig. 8.6(e)), crack-pattern development and failure mode (Fig. 8.8(b) vs. Fig. 8.8(c)). However, F.HC5.H.70 which was expected to show a better fire performance compared to F.HC4.H.40 due to a higher fiber content (70 kg/m^3 vs. 40 kg/m^3) failed 10 min earlier than F.HC4.H.40. This could possibly be due to a higher load that F.HC5.H.70 had to withstand during fire exposure (267 kN compared to 208 kN in F.HC4.H.40). This was because all the specimens were subjected to a load level of 40% of their ambient capacity while shear strength at room temperature of F.HC5.H.70 was 28.5% higher than that of F.HC4.H.40 (as defined by the ambient temperature test). Images from SEM as presented in Fig. 8.27 show that microcracks occurred along the steel fiber/matrix interface and many radial cracks formed after the sample was heated up to 200°C . This phenomenon took place due to thermal incompatibility between the steel fiber and the concrete matrix,

weakening the bond between them. As temperature increased, the fiber matrix bond strength continually decreased, reducing the effectiveness of steel fibers. Thus, under elevated temperatures, F.HC5.H.70 subjected to a higher applied load could fail earlier compared to F.HC4.H.40. However, it is worth mentioning that the melting temperature of steel fiber is very high (around 1370 °C). Therefore, even debonding phenomenon occurred during the fire tests, the steel fibers remained in the concrete matrix during heating time and contributed to shear resistance. This explains the superior performance of the three slabs using steel fibers compared to the three slabs without steel fibers. Most importantly, the experimental results suggest that the use of 40 kg/m³ hooked steel fibers (0.51% by volume) should be sufficient to prevent shear failure in hollow-core slabs in the event of fire.

Fig. 8.27 Microstructural observation in F.HC4.H.40 sample using SEM images

To evaluate the effectiveness of different fiber types on fire behavior of hollow-core slabs, test results of F.HC5.H.70 and F.HC6.S.70 were compared. As shown in Section 8.3, while web-shear and flexural-shear failure were prevented with the use of hooked steel fibers, flexural-shear failure occurred in the slab with the use of high-strength/straight fibers (F.HC6.S.70). It was also shown that F.HC6.S.70 failed 22 min earlier compared to F.HC5.H.70, indicating that hooked steel fibers had greater strength enhancement for PCHC slabs compared to high-strength/straight fibers. This was expected because fiber matrix bond strength entirely comes from the friction between the concrete and the steel fibers (Dinh 2009). Thus, the hooked steel fibers with a higher aspect ratio (80 vs. 62) and deformed ends could produce stronger bond, resulting in

higher shear resistance compared to straight steel fibers. Furthermore, the deformed ends of hooked steel fibers were beneficial in resisting pull-out forces when fiber matrix bond strength was degraded due to elevated temperatures. Test results showed that with the same fiber content of 0.89%, under fire conditions, F.HC5.H.70 with hooked fibers exhibited better shear resistance compared to F.HC6.S.70 of which high strength/straight fibers were used.

8.6 Conclusions

Six hollow-core specimens using PP and steel fibers (hooked and straight fibers) with different fiber contents were cast using the extrusion process. They were tested under fire conditions to examine shear behavior of fire-exposed FRC hollow-core slabs. The following conclusions are drawn:

1. Structural response of hollow-core slabs with PP fibers was very similar to that of the control slab without any fibers. Web-shear failure occurred, and the slabs suddenly failed at an early stage without any warnings.
2. Specimens with steel fibers exhibited superior performance under fire conditions compared to the companion slab without any fibers. Failure mode changed from web-shear to flexural-shear or purely bending failure. The specimens with steel fibers also exhibited greater ductility and developed large crack openings and deformations before failure. With the use of 40 kg/m^3 (0.51% by volume) of hooked steel fibers, web-shear failure (brittle type) in fire-exposed PCHC slabs was completely avoided.
3. Hooked steel fibers showed more favorable effect in shear resistance of hollow-core slabs under fire conditions compared to high-strength/straight steel fibers. Slabs with hooked steel fibers could sustain the same level of load in longer durations and had larger deformations before failure. In addition, the two specimens with hooked steel fibers failed in a more ductile mode with yielding and rupture of strands (flexural mode), while shear cracks still occurred in the specimen with high strength/straight steel fibers.

4. Severe spalling occurred in all the four specimens without PP fibers and with steel fibers whereas the phenomenon was not observed in the two specimens using PP fibers. The formation of microcracks in the radial direction of fiber tunnels created inter-connected fiber-tunnel-microcrack network, which provided pathways for releasing water vapor and increased permeability of concrete as shown in Zhang, Dasari et al. (2018) and Li, Tan et al. (2019). This network was observed at micro-level using SEM images in this study. Experimental results showed that the use of 1 kg/m^3 of PP fibers was sufficient to prevent explosive spalling of hollow-core slabs with a mean strength of 48 MPa.
5. The developed FE models were capable of predicting temperature development and fire responses of PCHC slabs with and without steel fibers. Numerical analysis showed that under loading-cum-heating effect, concrete webs of PCHC slabs showed tensile stresses in vertical, transverse, and longitudinal directions, making them more vulnerable under fire effect. It was shown that fire-induced tensile stresses in the concrete webs was the major factor for web-shear failure of PCHC slabs.

CHAPTER 9 CONCLUSIONS AND RECOMMENDATIONS

9.1 Summary

Shear failure, especially web-shear failure, is critical for precast/prestressed concrete hollow-core (PCHC) slabs with depths more than 300 mm at both ambient and in the event of fire. Deep PCHC slabs could fail in web-shear mode at loads lower than those predicted by some design codes. Under fire conditions, deep PCHC slabs subjected to shear commonly could not sustain their designed fire resistance and failed at an early stage of fire exposure as shown in the literature. For these reasons, the research program presented in this thesis was carried out aiming to:

1. Gain in-depth understanding of web-shear behavior of deep PCHC slabs at ambient and in the event of fire.
2. Develop robust finite element (FE) models which can simulate web-shear behavior of PCHC slabs at room and elevated temperatures. These numerical models, in turn, can be used to further investigate and extend the understanding of failure mechanisms of deep PCHC slabs.
3. Investigate shear responses of PCHC slabs incorporating polypropylene (PP) as well as hooked and high-strength/straight steel fibers. The effectiveness and the potential use of such fibers on enhancing shear strength of PCHC slabs at ambient and under fire conditions can then be quantified.
4. Evaluate the validity of ACI 318-14, Eurocode 2, EN 1168 in predicting web-shear strength of deep PCHC slabs.
5. Develop semi-analytical models to predict shear capacity of PCHC slabs with and without steel fibers.

To achieve the stated objectives, an experimental program consisting of 27 full-scale tests on 21 hollow-core specimens with and without fibers, divided into 4 series, was

carried out. The test data were used to investigate shear behavior of PCHC slabs. Moreover, they were used to verify FE models developed to simulate shear behavior of PCHC slabs at ambient and in the event of fire. Parametric studies based on the verified FE models were subsequently performed. Results and observations from the experimental and numerical studies were also used to develop two semi-analytical models to predict web-shear strength of conventional PCHC slabs and shear strength of PCHC slabs with steel fibers.

9.2 Conclusions

Based on the experimental, numerical and theoretical investigations presented in this thesis, the following key conclusions are drawn:

(i) Experimental and numerical studies on conventional PCHC slabs at ambient (Series I) – significant findings

1. ACI 318-14, Eurocode 2, and EN 1168:2005+A3:2011 overpredict shear capacity of some of the test specimens.
2. ABAQUS/explicit (2014) and The Concrete Damage Plasticity model (CDP model) can be used to simulate web-shear behavior of PCHC slabs. However, as numerical studies show that web-shear resistance is highly sensitive to dilation angle (one of the four plastic parameters in the CDP mode), the selection of dilation angle in numerical studies must be calibrated by actual test data to provide sensible predictions.
3. The choice of angular or smoother-surface void shapes does not lead to a noticeable difference in web-shear capacity. However, concrete strength plays an important role in web-shear performance. Moreover, web-shear strength increases with web thickness.
4. A high level of prestressing force increases additional shear stresses in concrete webs due to changes in strand stress within the transmission length region. Additionally, it leads to higher compressive forces in concrete diagonal struts.

The presence of such a higher compression force reduces the resistance in tension of concrete webs in the direction of maximum principal tensile stress. As a result, the web-shear strength of PCHC slabs decreases with increasing prestressing force.

5. A high level of prestressing force may be responsible for the brittle failure of deep PCHC units since these members are commonly designed with a higher level of prestressing force as a result of congested strands.

(ii) Experimental and numerical studies on conventional PCHC slabs under fire conditions (Series II) – significant findings

1. Under a load level of 40%, deep PCHC slabs subjected to shear failed in web-shear mode when furnace temperature had not yet reached 950°C, showing premature failure.
2. Four types of cracks were sequentially observed in the structural fire shear tests, i.e. splitting, longitudinal, thermal, and diagonal web-shear cracks. The formation of web-shear cracks substantially reduced the load-bearing capacity of PCHC slabs, directly inducing brittle failure.
3. ABAQUS (2014) and the numerical techniques adopted in this study can be used to simulate thermal and structural responses of PCHC slabs under fire conditions. Thermal, longitudinal and web-shear cracks were well captured by the numerical models. However, splitting cracks and anchorage failure could not be simulated due to an assumption of perfect bonding between the concrete and the strand. This is a limitation of the developed FE models.
4. With the same load level of 40%, web-shear resistance of the hollow-core units decreased as slab thickness increased. Besides, fire resistance decreased with increasing load level. Furthermore, temperature-induced tensile stresses in concrete webs, instead of temperature-induced degradation in strength and stiffness of concrete and prestressing steels, governed the web-shear behavior of deep PCHC slabs.

5. With an applied load level of 40%, the prescriptive method in Eurocode 2 which is based on the axis distance and thickness of concrete members gives unconservative prediction for fire resistance of deep PCHC slabs subjected to shear. In contrast, the formula in EN 1168 provides safe predictions for shear strength of PCHC slabs subjected to fire.

(iii) Investigations on fiber-reinforced concrete (FRC) hollow-core slabs at ambient (Series III) – significant findings

1. It was feasible to include PP fibers with a content up to 0.22% by volume or steel fibers with a volumetric content up to 0.89% in very dry (nearly zero-slump) concrete in the extrusion method.
2. It was experimentally observed that there were moderate increases in shear strength with the use of PP fibers while substantial increases in shear resistance were observed with the use of steel fibers. In addition, shear strength increased with steel-fiber content.
3. With the same steel fiber content of 0.89%, the specimens with high-strength/straight fibers exhibited superior performance in shear resistance compared to the specimens using hooked steel fibers.
4. With the inclusion of steel fibers up to 0.89% in PCHC slabs, in some instances, failure mode changed from web-shear to flexural-shear failure.
5. The *fib* 2010 model code provides an over-conservatism for shear strength of SFRC hollow-core slabs.

(iv) Investigations on fiber-reinforced concrete (FRC) hollow-core slabs under fire conditions (Series IV) – significant findings

1. Fire responses of PCHC slabs with PP fibers up to 0.22% by volume were similar to those of conventional PCHC slabs without any fibers. Post-cracking behavior was not improved with the use of PP fibers. All the slabs with PP fibers and without any fibers suddenly failed in web-shear mode at an early stage of fire

exposure, showing brittle failure. However, it is shown that resistance of concrete to explosive spalling was improved with the use of PP fibers.

2. Hollow-core slabs with steel fibers exhibited superior performance under fire conditions compared to conventional PCHC slabs. With the use of steel fibers up to 0.89% by volume, resistance to load and elevated temperatures of PCHC slabs was substantially enhanced. In addition, ductility and toughness of steel-fiber specimens subjected to elevated temperatures were also significantly improved. Furthermore, the failure mode shifted from web-shear to flexural-shear or flexural failure with the use of steel fibers.
3. Hooked steel fibers exhibited a more favorable effect on shear resistance of hollow-core slabs under fire conditions compared to high-strength/straight steel fibers. The two specimens with hooked steel fibers failed in a more ductile mode with yielding and rupture of strands (flexural mode), while shear cracks still occurred in the specimen with high strength/straight steel fibers.
4. The developed FE models using ABAQUS (2014) presented in Chapter 6 can be used to predict shear responses of fire-exposed PCHC slabs with and without PP fibers. Besides, behavior of SFRC hollow-core slabs subjected to fire can be simulated in ABAQUS (2014) with the incorporation of the *fib* 2010 constitutive model for steel-fiber-reinforced concrete in tension. However, fire-induced spalling could not numerically be simulated. This is another limitation of the developed FE models.
5. Numerical analysis showed that under loading-cum-heating effect, concrete webs of PCHC slabs were subjected to tensile stresses in vertical, transverse, and longitudinal directions, making them much more vulnerable under the combined effect. The numerical analysis further confirmed the finding from Chapter 6 that temperature-induced tensile stresses in the concrete webs played an important role in web-shear failure of conventional PCHC slabs. It is also shown that when steel fibers were added, tensile strength of concrete was considerably enhanced. Moreover, with random distribution, the fibers were able to resist tensile stresses

in the three directions. As a result, web-shear cracks could not be generated. The SFRC slabs failed in other failure modes (flexural-shear or flexure) which were triggered from the slab bottom due to degradation in strength of the concrete and the pretensioned strands due to elevated temperatures.

(v) The proposed semi-analytical models

Two semi-analytical models were developed in this study. The first model (presented in Chapter 5) was developed to predict web-shear strength of conventional PCHC slabs. Besides considering (1) the actual critical locations where web-shear cracks are initiated and (2) the additional tensile stresses due to changes of prestressing force within the transmission length, which are based on Yang (1994)'s approach, the model adopts a more rational failure criterion from Kupfer and Gerstle (1973) for concrete in biaxial principal compression-tension stress state. The model was verified using a database of 47 PCHC slabs from this study (2 tests) and from the literature (45 tests). It is shown that compared to ACI 318-14, Eurocode 2, and Yang's method, the proposed model provides the most reasonable and safe predictions for web-shear strength of PCHC slabs.

When steel fibers were added, the failure mode of PCHC slabs commonly shifts from web-shear to flexural-shear mode as shown in Chapter 7. For this reason, the second semi-analytical formula (presented in Chapter 7) which is based on flexural-shear failure was developed for SFRC hollow-core slabs. The proposed formula takes into account the contribution of concrete in compression zone, prestress, steel fibers, and arching action to shear resistance. The formula was verified using a database of 45 shear tests (6 slabs with steel fibers from this research program and another 39 SFRC hollow-core slabs from the literature). The verification shows that the proposed equation is capable of predicting shear strength of SFR PCHC slabs with reasonable accuracy.

9.3 Recommendations

9.3.1 Recommendations for practice

Based on experimental, numerical, and theoretical investigations from this research, the following recommendations are made for practice:

1. The prescriptive method in Eurocode 2 should not be used for fire design of deep PCHC slabs subjected to shear. This recommendation is based on numerical observations from Series II.
2. As hooked steel fibers with a length of 60 mm, an aspect ratio of 80, and a volume fraction of 0.89% were used, poor concrete compaction was observed at some locations, causing a reduction in shear resistance (Chapter 7). Therefore, it is recommended not to use a combination of 60 mm length hooked steel fibers with an aspect ratio of 80 at a volume fraction of 0.89% in the extrusion method.
3. It is shown by the author and other researchers (Peaston and Paine 1999, Simasathien and Chao 2015) that the effectiveness of steel fibers is greater when they are used in hollow-core slabs than in beams. This is due to the thin webs of hollow-core slabs and compacting process of the extrusion method that tend to orientate the fibers vertically in the concrete webs. Therefore, fiber content between 0.5 to 0.89% should be sufficient to enhance shear resistance of hollow-core slabs. Furthermore, the use of hooked steel fibers with similar contents and proper aspect ratios should help to prevent brittle, premature failure in PCHC slabs in the event of fire.
4. The use of 1 kg/m³ of PP fibers should be sufficient to prevent explosive spalling of PCHC slabs with a mean strength of 48 MPa.
5. As the semi-analytical model presented in Chapter 5 provides the most reasonable and safe predictions for web-shear strength of PCHC slabs, it is recommended to replace the current models in EC2 and ACI code for designing PCHC slabs against web-shear failure.

6. The proposed formula in Chapter 7 is recommended to predict shear strength of SFR PCHC slabs. However, as the majority of the tests in the database (43 out of 45 tests) used to verify the formula were conducted on hooked-steel-fiber-reinforced PCHC slabs, it is recommended to limit the use of the formula for only hooked SFR PCHC slabs.

9.3.2 Recommendations for future work

Experimental study on steel-fiber-reinforced PCHC slabs

As shown in Chapter 7, with the same fiber content of 0.89% by volume, the specimens using high-strength/straight fibers (a length of 13 mm and an aspect ratio of 62) exhibited superior performance in shear resistance compared to the specimens using hooked steel fibers (a length of 60 mm and an aspect ratio of 80). This is possibly because poor concrete compaction was observed in some locations in the slabs with the hooked steel fibers, causing a reduction in shear resistance. However, the experimental data in Chapter 8 showed that hooked steel fibers showed a more favorable effect on enhancing shear resistance under fire conditions. Thus, hooked steel fibers with a length shorter than 60 mm with aspect ratio ranging from 62 to 80 can be incorporated in a further experimental program. The use of shorter fiber lengths and finer fiber diameters could help to overcome consistency and compactness problems of using hooked steel fibers in nearly-zero-slump concrete in the extrusion process, thus enhancing shear strength at both ambient and fire conditions.

Numerical modeling of PCHC slabs under fire conditions

Although the FE models presented in Chapters 6 and 8 could simulate thermal and structural responses of PCHC slabs in the event of fire, there are still some features needed to be improved. First, the developed models assumed perfect bonding between the concrete and the prestressing strands. Therefore, splitting cracks and anchorage failure could not be simulated. Furthermore, explosive spalling of concrete could not be predicted in the developed numerical models. Thus, additional work is necessary to

develop a more sophisticated FE model incorporating bond-slip models for prestressing steels and accounting for temperature-induced spalling of concrete.

Analytical model for shear strength of PCHC slabs subjected to fire

It is shown in Chapter 6 that EN 1168 provides safe predictions for shear strength of PCHC slabs subjected to fire. However, the shear formula in EN 1168 only accounts for degradation in strength of concrete, prestressing strands and the bond between them. It does not take into account the effect of temperature-induced tensile stresses in concrete which was shown to play an important role and to govern web-shear behavior of PCHC slabs under fire conditions. Therefore, further work is required to improve the approach for more rational predictions.

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APPENDIX A - CHAPTER 3

Fig. A1: Extruder and casting bed in the precast plant

Fig. A2: Rotating augers in an extruder (forming shapes of hollow-cores and compacting concrete)

Fig. A3: Concrete from an onsite batching plant was poured into an extruder from the top

Fig. A4: Casting concrete using the extrusion method

APPENDIX B - CHAPTER 5

APPENDIX B1: MATLAB CODES TO DETERMINE MAXIMUM SHEAR STRESS

```
[sig_1,sig_2,tau,]=solve('sig_1/ft=1+0.8*sig_2/fc','sig_1=sig_x/2+(sig_x^2/4+tau^2)^(1/2)', 'sig_2=sig_x/2-(sig_x^2/4+tau^2)^(1/2)')
simplify(tau)

pretty(tau)

ans =

-(2.2360679774997896964091736687313*(fcu*ftu*(ftu + sig_x)*(5.0*fcu - 4.0*sig_x))^(1/2))/(5.0*fcu + 4.0*ftu)
(2.2360679774997896964091736687313*(fcu*ftu*(ftu + sig_x)*(5.0*fcu - 4.0*sig_x))^(1/2))/(5.0*fcu + 4.0*ftu)

>> pretty(tau)
/  sqrt(fcu ftu (ftu + sig_x) (5.0 fcu - 4.0 sig_x)) 2.2360679774997896964091736687313
\
| -----
|                                     5.0 fcu + 4.0 ftu
|
|
| sqrt(fcu ftu (ftu + sig_x) (5.0 fcu - 4.0 sig_x)) 2.2360679774997896964091736687313
| -----
|
|                                     5.0 fcu + 4.0 ftu
\
/

>>
```

APPENDIX B2: DATABASE FOR MODEL VERIFICATION (Eq. 5.20)

45 test results (from Test ID. 31.200 to 201.500) were compiled by Pajari (2005); The last two test in the database (HC2.400 and HC3.500) are from this study.

Table B1: Test slab, concrete strength, geometric properties

Test ID.	Slab depth (mm)	Compressive strength		Total web width (mm)	Axis distance (mm)	Cross sectional area (mm ²)	I_y (mm ⁴)
		$f_{c,mean}$ (MPa)	Standard deviation (MPa)				
31.200	200	54.8	3.8	266.17	45	2.1E+05	4.4E+09
33.200	200	52.8	3.2	300	52.5	2.6E+05	8.2E+09
40.200	200	76.5	3.8	239	39	1.2E+05	6.0E+08
63.200	200	56.6	2.5	238	40	1.2E+05	6.0E+08
98.265	265	71.3	4.2	293	39	1.3E+05	6.2E+08
104.265	265	47.6	4.0	262	38	1.3E+05	6.2E+08
107.265	265	47.9	3.1	228	36	1.7E+05	1.5E+09
109.265	265	56.7	3.0	244	45	1.7E+05	1.5E+09
110.265	265	56.5	2.6	239	39	1.7E+05	1.5E+09
115.265	265	63.2	4.9	242	39	1.7E+05	1.5E+09
501.265	265	67.1	2.4	220	32	1.6E+05	1.5E+09
502.265	265	67.1	2.4	215	40	1.6E+05	1.5E+09
503.265	265	72.0	4.9	224	41	1.7E+05	1.5E+09
504.265	265	72.0	4.9	224	41	1.7E+05	1.5E+09
505.265	265	70.8	4.3	221	34	1.7E+05	1.5E+09
506.265	265	70.8	4.3	221	34	1.7E+05	1.5E+09
507.265	265	68.0	8.1	216	35	1.7E+05	1.5E+09
508.265	265	68.0	8.1	216	35	1.7E+05	1.5E+09
509.265	265	65.8	2.1	218	37	1.7E+05	1.5E+09
510.265	265	65.8	2.1	218	37	1.7E+05	1.5E+09
511.265	265	67.8	2.5	217	36	1.7E+05	1.5E+09
512.265	265	67.8	2.9	217	36	1.7E+05	1.5E+09
133.320	320	68.0	1.9	221	40	1.7E+05	1.5E+09
134.320	320	68.0	1.9	223	39	1.7E+05	1.5E+09
146.320	320	57.0	3.6	261	48	2.4E+05	2.9E+09
151.320	320	66.8	5.5	243	47	2.4E+05	2.9E+09
514.320	320	61.8	2.4	256	33	2.0E+05	2.6E+09
515.320	320	61.8	2.4	270	48	2.0E+05	2.6E+09
516.320	320	64.3	2.6	309	43	1.9E+05	2.5E+09
517.320	320	64.8	1.2	311	42	1.9E+05	2.5E+09

Test ID.	Slab depth (mm)	Compressive strength		Total web width (mm)	Axis distance (mm)	Cross sectional area (mm ²)	I_y (mm ⁴)
		$f_{c,mean}$ (MPa)	Standard deviation (MPa)				
160.370	370	72.2	2.1	289	38	1.8E+05	2.4E+09
161.370	370	72.2	2.1	287	39	1.8E+05	2.4E+09
162.400	400	65.2	5.0	272	40	2.4E+05	4.1E+09
178.400	400	71.5	4.0	276	39	2.4E+05	4.1E+09
188.400	400	55.5	2.9	284	53	2.1E+05	4.4E+09
518.400	400	67.3	1.7	286	38	2.2E+05	4.5E+09
519.400	400	67.3	1.7	285	39	2.2E+05	4.5E+09
520.400	400	64.2	2.5	293	41	2.1E+05	4.4E+09
521.400	400	64.2	2.5	293	41	2.1E+05	4.4E+09
191.500	500	70.8	1.1	291	36	2.1E+05	4.4E+09
193.500	500	70.8	1.1	291	36	2.1E+05	4.4E+09
198.500	500	76.4	2.6	325	53	3.0E+05	9.3E+09
199.500	500	74.7	5.3	325	56	3.0E+05	9.3E+09
200.500	500	74.7	5.3	324	47	3.0E+05	9.3E+09
201.500	500	74.7	5.3	326	52	3.0E+05	9.3E+09
HC2.400	400	57.3	4.0	327	53	3.0E+05	9.3E+09
HC3.500	500	56.0	4.0	322	51	3.0E+05	9.3E+09

Table B2: Test configurations and strand type

Test ID.	a/d	Bearing width (mm)	t^* (mm)	No. of strand	Strand diameter (mm)	Strand area (mm ²)	Jacking prestress (MPa)
31.200	5.7	40	20	7	12.5	93	1100
33.200	4.3	40	20	7	12.5	93	1100
40.200	5.4	40	20	7	12.5	93	1100
63.200	3.4	40	20	7	12.5	93	1000
98.265	3.2	40	20	10	12.5	93	1100
104.265	3.1	40	20	4	12.5	93	1000
107.265	3.1	40	20	4	12.5	93	1000
109.265	3.0	40	20	10	12.5	93	1000
110.265	3.0	40	20	8	12.5	93	1000
115.265	3.0	40	20	6	12.5	93	1000
501.265	4.3	75	38	10	12.5	93	1100
502.265	4.3	75	38	10	12.5	93	1100
503.265	4.2	60	30	10	12.5	93	950
504.265	4.2	60	30	10	12.5	93	950
505.265	4.2	60	30	10	12.5	93	950
506.265	4.2	60	30	10	12.5	93	950
507.265	4.3	60	30	10	12.5	93	950
508.265	4.3	60	30	10	12.5	93	950
509.265	4.2	60	30	10	12.5	93	950
510.265	4.2	60	30	10	12.5	93	950
511.265	5.2	60	30	10	12.5	93	950
512.265	5.2	60	30	10	12.5	93	950
133.320	3.0	60	30	12	12.5	93	1100
134.320	3.6	60	30	12	12.5	93	1100
146.320	2.8	40	20	6	12.5	93	1000
151.320	3.0	40	20	9	12.5	93	1100
514.320	4.2	80	40	13	12.5	93	1100
515.320	4.2	80	40	13	12.5	93	1100
516.320	2.7	80	40	11	12.5	93	1000
517.320	2.7	80	40	11	12.5	93	1000
160.370	2.9	60	30	12	12.5	93	1000
161.370	3.5	60	30	12	12.5	93	1000
162.400	4.0	80	40	12	12.5	93	1100

Test ID.	a/d	Bearing width (mm)	t^* (mm)	No. of strand	Strand diameter (mm)	Strand area (mm ²)	Jacking prestress (MPa)
178.400	2.9	80	40	13	12.5	93	1100
188.400	3.0	80	40	9	12.5	93	1000
518.400	3.2	100	50	13	12.5	93	1000
519.400	3.2	100	50	13	12.5	93	1000
520.400	2.7	75	38	11	12.5	93	1100
521.400	2.7	75	38	11	12.5	93	1100
191.500	3.1	80	40	16	12.5	93	1000
193.500	3.0	80	40	16	12.5	93	1000
198.500	3.3	80	40	16	12.5	93	1000
199.500	3.0	80	40	21	12.5	93	1000
200.500	3.3	80	40	21	12.5	93	1000
201.500	3.6	80	40	21	12.5	93	1000
HC2.400	2.3	100	150	11	9.6 & 12.9	55 & 99	1150
HC3.500	1.8	100	150	11	12.9	99	1150

* t : distance from slab edge to the center of bearing support

Table B3: Shear-strength comparisons: Experimental results vs. formulas using mean strengths of concrete

Test ID.	V_{exp} (kN)	V_{pre} (kN)				V_{exp} / V_{pre}			
		Proposed model	Yang's (1994)	EC2	ACI 318-14	Proposed model	Yang's (1994)	EC2	ACI 318- 14
31.200	90.2	143.9	126.7	143.9	96.0	0.63	0.71	0.63	0.94
33.200	115.6	140.1	123.3	141.0	93.5	0.82	0.94	0.82	1.24
40.200	106.0	199.0	190.6	216.0	135.3	0.53	0.56	0.49	0.78
63.200	137.5	157.3	146.1	163.8	105.4	0.87	0.94	0.84	1.30
98.265	219.4	213.3	218.8	220.5	149.9	1.03	1.00	1.00	1.46
104.265	136.0	168.1	148.1	162.4	115.8	0.81	0.92	0.84	1.17
107.265	134.2	171.7	152.7	164.1	116.8	0.78	0.88	0.82	1.15
109.265	188.2	203.3	198.3	204.6	140.4	0.93	0.95	0.92	1.34
110.265	194.9	195.9	182.2	184.7	128.4	1.00	1.07	1.06	1.52
115.265	176.1	186.4	170.7	181.9	123.4	0.94	1.03	0.97	1.43
501.265	213.4	205.6	212.6	221.2	144.6	1.04	1.00	0.96	1.48
502.265	205.0	205.6	212.6	221.2	144.6	1.00	0.96	0.93	1.42
503.265	208.0	208.4	219.5	217.5	146.4	1.00	0.95	0.96	1.42
504.265	177.8	208.4	219.5	217.5	146.4	0.85	0.81	0.82	1.21
505.265	213.3	202.1	211.7	211.2	141.5	1.06	1.01	1.01	1.51
506.265	216.5	202.1	211.7	211.2	141.5	1.07	1.02	1.03	1.53
507.265	195.8	191.6	191.7	194.3	139.1	1.02	1.02	1.01	1.41
508.265	186.7	191.6	191.7	194.3	139.1	0.97	0.97	0.96	1.34
509.265	188.4	199.7	209.0	210.0	137.1	0.94	0.90	0.90	1.37
510.265	200.7	199.7	209.0	210.0	137.1	1.00	0.96	0.96	1.46
511.265	225.4	200.9	209.1	216.4	139.0	1.12	1.08	1.04	1.62
512.265	227.0	203.2	212.1	218.0	140.9	1.12	1.07	1.04	1.61
133.320	296.4	290.0	298.3	301.6	203.2	1.02	0.99	0.98	1.46
134.320	290.5	270.9	280.2	280.9	189.9	1.07	1.04	1.03	1.53
146.320	213.7	252.9	214.9	245.1	177.6	0.84	0.99	0.87	1.20
151.320	255.3	276.6	276.9	289.3	201.4	0.92	0.92	0.88	1.27
514.320	297.1	268.3	318.6	371.2	127.2	1.11	0.93	0.80	2.34
515.320	293.7	271.4	320.7	373.6	128.5	1.08	0.92	0.79	2.29
516.320	232.5	256.0	312.4	353.0	117.9	0.91	0.74	0.66	1.97
517.320	202.7	256.4	315.4	358.0	117.0	0.79	0.64	0.57	1.73
160.370	306.2	303.3	367.3	389.4	131.4	1.01	0.83	0.79	2.33
161.370	282.2	308.4	374.1	395.2	133.8	0.92	0.75	0.71	2.11
162.400	314.1	281.6	337.7	423.8	147.8	1.12	0.93	0.74	2.12

Test ID.	V_{exp} (kN)	V_{pre} (kN)				V_{exp} / V_{pre}			
		Proposed model	Yang's (1994)	EC2	ACI 318-14	Proposed model	Yang's (1994)	EC2	ACI 318- 14
178.400	291.3	367.8	434.5	465.7	163.6	0.79	0.67	0.63	1.78
188.400	292.9	314.3	354.3	375.7	133.8	0.93	0.83	0.78	2.19
518.400	394.9	316.3	401.8	490.0	162.4	1.25	0.98	0.81	2.43
519.400	444.0	316.3	401.8	490.0	162.4	1.40	1.10	0.91	2.73
520.400	345.2	305.6	381.0	450.0	155.1	1.13	0.91	0.77	2.23
521.400	391.2	305.6	381.0	450.0	155.1	1.28	1.03	0.87	2.52
191.500	363.2	429.0	539.9	679.4	225.6	0.85	0.67	0.53	1.61
193.500	415.0	423.4	547.8	702.6	224.1	0.98	0.76	0.59	1.85
198.500	471.2	444.5	562.3	687.5	234.9	1.06	0.84	0.69	2.01
199.500	557.6	477.5	538.1	700.1	246.7	1.17	1.04	0.80	2.26
200.500	514.8	476.0	536.2	704.0	246.9	1.08	0.96	0.73	2.08
201.500	491.4	476.1	534.8	692.1	244.2	1.03	0.92	0.71	2.01
HC2.400	266.6	295.5	351.1	458.3	144.6	0.90	0.76	0.58	1.84
HC3.500	593.4	379.2	434.9	630.0	199.1	1.56	1.36	0.94	2.98
Mean						0.99	0.92	0.83	1.71
CoV						0.18	0.16	0.18	0.28

Table B4: Shear-strength comparisons: Experimental results vs. formulas using characteristic strengths of concrete

Test ID.	V_{exp} (kN)	V_{pre} (kN)				V_{exp} / V_{pre}			
		Proposed model	Yang's (1994)	EC2	ACI 318- 14	Proposed model	Yang's (1994)	EC2	ACI 318- 14
31.200	90.2	101.9	92.0	105.5	89.7	0.89	0.98	0.85	1.01
33.200	115.6	98.9	89.3	103.4	87.1	1.17	1.29	1.12	1.33
40.200	106.0	141.7	140.4	158.4	128.8	0.75	0.75	0.67	0.82
63.200	137.5	111.8	107.2	119.7	98.6	1.23	1.28	1.15	1.40
98.265	219.4	156.3	165.4	162.9	142.5	1.40	1.33	1.35	1.54
104.265	136.0	119.8	107.4	116.8	106.3	1.14	1.27	1.16	1.28
107.265	134.2	123.1	111.6	118.1	107.4	1.09	1.20	1.14	1.25
109.265	188.2	148.0	148.9	151.0	131.7	1.27	1.26	1.25	1.43
110.265	194.9	143.7	136.7	135.4	120.2	1.36	1.43	1.44	1.62
115.265	176.1	134.2	125.1	132.2	116.1	1.31	1.41	1.33	1.52
501.265	213.4	150.0	160.4	164.8	137.3	1.42	1.33	1.29	1.55
502.265	205.0	150.0	160.4	164.8	137.3	1.37	1.28	1.24	1.49
503.265	208.0	153.2	167.5	161.5	139.2	1.36	1.24	1.29	1.49
504.265	177.8	153.2	167.5	161.5	139.2	1.16	1.06	1.10	1.28
505.265	213.3	148.3	161.1	156.7	134.4	1.44	1.32	1.36	1.59
506.265	216.5	148.3	161.1	156.7	134.4	1.46	1.34	1.38	1.61
507.265	195.8	140.2	145.1	144.1	131.9	1.40	1.35	1.36	1.48
508.265	186.7	140.2	145.1	144.1	131.9	1.33	1.29	1.30	1.42
509.265	188.4	146.3	158.7	155.8	129.8	1.29	1.19	1.21	1.45
510.265	200.7	146.3	158.7	155.8	129.8	1.37	1.26	1.29	1.55
511.265	225.4	146.0	157.4	160.7	131.8	1.54	1.43	1.40	1.71
512.265	227.0	148.0	160.1	161.9	133.5	1.53	1.42	1.40	1.70
133.320	296.4	213.0	225.0	223.6	192.8	1.39	1.32	1.33	1.54
134.320	290.5	199.2	211.6	208.2	180.2	1.46	1.37	1.40	1.61
146.320	213.7	185.1	182.0	177.8	165.8	1.15	1.17	1.20	1.29
151.320	255.3	201.0	208.8	212.0	190.6	1.27	1.22	1.20	1.34
514.320	297.1	210.2	233.8	279.2	120.6	1.41	1.27	1.06	2.46
515.320	293.7	212.9	235.4	281.0	121.9	1.38	1.25	1.05	2.41
516.320	232.5	198.6	230.2	264.4	111.8	1.17	1.01	0.88	2.08
517.320	202.7	198.5	232.1	268.3	111.0	1.02	0.87	0.76	1.83
160.370	306.2	236.8	274.4	290.4	125.1	1.29	1.12	1.05	2.45
161.370	282.2	240.9	279.9	294.7	127.3	1.17	1.01	0.96	2.22
162.400	314.1	223.5	240.0	318.0	140.5	1.41	1.31	0.99	2.24

Test ID.	V_{exp} (kN)	V_{pre} (kN)				V_{exp} / V_{pre}			
		Proposed model	Yang's (1994)	EC2	ACI 318- 14	Proposed model	Yang's (1994)	EC2	ACI 318- 14
178.400	291.3	291.2	324.2	351.0	156.3	1.00	0.90	0.83	1.86
188.400	292.9	241.5	261.2	278.7	125.4	1.21	1.12	1.05	2.34
518.400	394.9	254.4	289.8	371.2	154.7	1.55	1.36	1.06	2.55
519.400	444.0	254.4	289.8	371.2	154.7	1.75	1.53	1.20	2.87
520.400	345.2	245.8	274.9	337.4	147.2	1.40	1.26	1.02	2.35
521.400	391.2	245.8	274.9	337.4	147.2	1.59	1.42	1.16	2.66
191.500	363.2	349.4	379.9	514.2	215.3	1.04	0.96	0.71	1.69
193.500	415.0	343.9	382.9	532.1	213.8	1.21	1.08	0.78	1.94
198.500	471.2	363.7	400.3	520.1	224.8	1.30	1.18	0.91	2.10
199.500	557.6	398.4	375.7	534.9	236.6	1.40	1.48	1.04	2.36
200.500	514.8	396.7	373.1	538.1	236.8	1.30	1.38	0.96	2.17
201.500	491.4	398.0	374.2	528.8	234.2	1.23	1.31	0.93	2.10
HC2.400	266.6	235.7	273.3	345.3	137.1	1.13	0.98	0.77	1.94
HC3.500	593.4	304.9	343.3	482.1	188.2	1.95	1.73	1.23	3.15
Mean						1.31	1.24	1.12	1.81
CoV						0.16	0.15	0.19	0.28

APPENDIX C - CHAPTER 6

Fig. C1 F.HC1.320 in Annex Lab (NTU) before setting up

Fig. C1 Test setup (F.HC1.320)

Fig. C3 Fire test setup (F.HC2.400)

Fig. C4 Smoke escaped during the fire test (F.HC2.400)

Fig. C5 Testing PCHC slabs under fire conditions (F.HC2.400)

Fig. C6 F.HC3.500 after testing

APPENDIX D - CHAPTER 7

Zsutty (1971)'s formula:

Average shear stress at failure of conventional beams without fibers:

$$\nu_u = 11.4e \left(f_c \rho \frac{d}{a} \right)^{1/3}$$

where $e = 1.0$ for $a/d > 2.5$, and 2.5 for d/a for $a/d \leq 2.5$; ρ is flexural reinforcement ratio.

Yoon-Keun Kwak and Jubum (2002)'s formula:

Average shear stress at failure of SFRC beams:

$$\nu_u = A e f_{spfc}^{exp1} \left(\rho \frac{d}{a} \right)^{exp2} + B \nu_b^{ex} \quad (\text{MPa})$$

where

$\nu_b = 0.41 \tau F$; F is fiber factor; $e = 1.0$ for $a/d > a/d_{transition}$ and $e = (a/d_{transition} \times d/a)$ for $a/d < a/d_{transition}$; $a/d_{transition} = 3.5$; $A = 2.1$; $B = 0.8$; $exp1 = 0.7$; $exp2 = 0.22$; and $exp3 = 0.97$.

Table D1 Database for shear predictions (Eq. 7.9) of SFRC hollow-core slabs

Author	Test No.	f'_c (cube strength) (MPa)	V_f (%)	L_f (mm)	D_f (mm)	ρ (%)	b_w (mm)	d (mm)	a/d	V_{exp} (kN)	V_{pre} (Eq.7.9) (kN)	$\frac{V_{exp}}{V_{pre}}$
This study	HC6.S.70.a	78.03	0.89	13	0.21	0.95	292	363	2.81	693.0	381.2	1.82
	HC6.S.70.a	78.03	0.89	13	0.21	0.95	292	363	2.81	604.0	381.2	1.58
	HC4.H.40.a	81.60	0.51	60	0.75	0.95	289	364	2.81	372.0	362.5	1.03
	HC4.H.40.b	81.60	0.51	60	0.75	0.95	289	364	2.81	406.0	362.5	1.12
	HC5.H.70.a	76.48	0.89	60	0.75	0.94	292	365	2.81	556.0	427.5	1.30
	HC5.H.70.b	76.48	0.89	60	0.75	0.94	292	365	2.81	444.0	427.5	1.04
Peaston et al. (1999)	F1A	86.5	0.50	30	0.5	1.57	271	153	2.80	180.0	144.9	1.24
	F4A	79.5	0.50	30	0.5	1.76	252	147	2.80	164.0	124.7	1.32
	F4B	79.5	0.50	30	0.5	1.76	252	147	2.80	161.0	124.7	1.29
	F6A	79.5	0.50	30	0.5	1.75	253	147	2.80	183.0	125.2	1.46
	F6B	79.5	0.50	30	0.5	1.75	253	147	2.80	174.0	125.2	1.39
	F1B	86.5	0.50	30	0.5	1.57	271	153	2.00	200.0	225.8	0.89
	F2A	86.5	0.50	30	0.5	1.59	269	152	2.00	219.0	222.7	0.98
	F2B	86.5	0.50	30	0.5	1.59	269	152	2.00	194.0	222.7	0.87
	F3A	79.5	0.50	30	0.5	1.44	293	154	2.00	159.0	234.4	0.68
	F3B	79.5	0.50	30	0.5	1.44	293	154	2.00	170.0	234.4	0.73
	F5A	91.2	1.00	30	0.5	1.75	250	149	2.00	232.0	237.4	0.98
	F5B	91.2	1.00	30	0.5	1.75	250	149	2.00	202.0	237.4	0.85
Cuenca and Serna (2012)	I-50-3.1	63	0.64	40	0.62	0.50	315	230	3.10	160.2	194.2	0.82
	I-50-3.9a	63	0.64	40	0.62	0.50	315	230	3.90	162.1	172.2	0.94
	I-50-3.9b	63	0.64	40	0.62	0.50	315	230	3.90	162.1	172.2	0.94
	I-50-4.3	63	0.64	40	0.62	0.50	315	230	4.30	152.6	164.2	0.93
	I-50-4.4	63	0.64	40	0.62	0.50	315	230	4.40	158.6	162.5	0.98

Author	Test No.	f'_c (cube strength) (MPa)	V_f (%)	L_f (mm)	D_f (mm)	ρ (%)	b_w (mm)	d (mm)	a/d	V_{exp} (kN)	V_{pre} (Eq.7.9) (kN)	$\frac{V_{exp}}{V_{pre}}$
	I-50-3.0a	63	0.64	40	0.62	0.50	315	230	3.00	173.0	197.8	0.87
	I-50-3.0b	63	0.64	40	0.62	0.50	315	230	3.00	200.6	197.8	1.01
	I-50-3.1c	63	0.64	40	0.62	0.50	315	230	3.10	189.4	194.2	0.98
	II-50-2.3	48	0.64	40	0.62	1.00	315	226	2.30	410.0	259.4	1.58
	II-50-3.4a	48	0.64	40	0.62	1.00	315	226	3.40	236.7	195.5	1.21
	II-50-3.4b	48	0.64	40	0.62	1.00	315	226	3.40	202.0	195.5	1.03
	II-70-2.3a	45	0.89	40	0.62	1.00	315	226	2.30	310.7	280.7	1.11
	II-70-2.3b	45	0.89	40	0.62	1.00	315	226	2.30	310.7	280.7	1.11
	II-70-3.4a	45	0.89	40	0.62	1.00	315	226	3.40	266.7	217.1	1.23
	II-70-3.4b	45	0.89	40	0.62	1.00	315	226	3.40	251.4	217.1	1.16
Dudnik et al. (2017)	12-B1-E-3.5d	78.1	0.38	30	0.56	0.92	336.55	254	3.50	233.5	210.2	1.11
	12-B1-W-3.5d	78.1	0.38	30	0.56	0.92	336.55	254	3.50	248.2	210.2	1.18
	12-B2-E-3.0d	78.1	0.38	30	0.56	0.92	336.55	254	3.00	295.3	231.1	1.28
	12-B2-W-3.0d	78.1	0.38	30	0.56	0.92	336.55	254	3.00	327.8	231.1	1.42
	12-C1-E-3.5d	92.1	0.50	30	0.56	0.92	336.55	254	3.50	277.1	240.0	1.15
	12-C1-W-3.5d	92.1	0.50	30	0.56	0.92	336.55	254	3.50	289.6	240.0	1.21
	12-C2-E-3.0d	92.1	0.50	30	0.56	0.92	336.55	254	3.00	320.3	264.2	1.21
	12-C2-W-3.0d	92.1	0.50	30	0.56	0.92	336.55	254	3.00	271.3	264.2	1.03
	12-D1-W-3.5d	65.8	0.76	30	0.56	0.92	336.55	254	3.50	245.1	232.6	1.05
	12-D2-E-3.0d	65.8	0.76	30	0.56	0.92	336.55	254	3.00	226.4	251.9	0.90
	Mean											1.12
	CoV											0.21

Table D2 Database for shear predictions of the 2010 *fib* model code

Author	Test No.	f_c (MPa)*	V_f (%)	L_f (mm)	D_f (mm)	ρ (%)	b_w (mm)	d (mm)	a/d	V_{exp} (kN)	V_{pre} (fib 2010) (kN)	$\frac{V_{exp}}{V_{pre}}$
This study	HC6.S.70.a	78.03	0.89	13	0.21	0.95	292	363	2.81	693.0	294.5	2.35
	HC6.S.70.a	78.03	0.89	13	0.21	0.95	292	363	2.81	604.0	294.5	2.05
	HC4.H.40.a	81.60	0.51	60	0.75	0.95	289	364	2.81	372.0	326.4	1.14
	HC4.H.40.b	81.60	0.51	60	0.75	0.95	289	364	2.81	406.0	326.4	1.24
	HC5.H.70.a	76.48	0.89	60	0.75	0.94	292	365	2.81	556.0	322.6	1.72
	HC5.H.70.b	76.48	0.89	60	0.75	0.94	292	365	2.81	444.0	322.6	1.38
Cuenca and Serna (2012)	I-50-3.1	63	0.64	40	0.62	0.50	315	230	3.10	160.2	136.1	1.18
	I-50-3.9a	63	0.64	40	0.62	0.50	315	230	3.90	162.1	136.1	1.19
	I-50-3.9b	63	0.64	40	0.62	0.50	315	230	3.90	162.1	136.1	1.19
	I-50-4.3	63	0.64	40	0.62	0.50	315	230	4.30	152.6	136.1	1.12
	I-50-4.4	63	0.64	40	0.62	0.50	315	230	4.40	158.6	136.1	1.17
	I-50-3.0a	63	0.64	40	0.62	0.50	315	230	3.00	173.0	136.1	1.27
	I-50-3.0b	63	0.64	40	0.62	0.50	315	230	3.00	200.6	136.1	1.47
	I-50-3.1c	63	0.64	40	0.62	0.50	315	230	3.10	189.4	136.1	1.39
	II-50-2.3	48	0.64	40	0.62	1.00	315	226	2.30	410.0	201.1	2.04
	II-50-3.4a	48	0.64	40	0.62	1.00	315	226	3.40	236.7	201.1	1.18
	II-50-3.4b	48	0.64	40	0.62	1.00	315	226	3.40	202.0	201.1	1.00
	II-70-2.3a	45	0.89	40	0.62	1.00	315	226	2.30	310.7	208.4	1.49
	II-70-2.3b	45	0.89	40	0.62	1.00	315	226	2.30	310.7	208.4	1.49
	II-70-3.4a	45	0.89	40	0.62	1.00	315	226	3.40	266.7	208.4	1.28
	II-70-3.4b	45	0.89	40	0.62	1.00	315	226	3.40	251.4	208.4	1.21
	Mean											1.41
	CoV											0.25

It should be noted that, in Peaston et al. (1999) and Dudnik et al. (2017) studies, flexural beam tests according to EN 14651 were not conducted. Therefore, these studies did not provide sufficient information and were excluded from the database.

APPENDIX E - CHAPTER 8

Fig. E1 Samples for scanning electron microscope (SEM)

a) Ready samples

b) Putting samples into the machine

Fig. E2 Preparations for obtaining SEM images