NUMERICAL SIMULATION OF AXIALLY LOADED CONCRETE FILLED HOLLOW STEEL SECTION COLUMNS AT ELEVATED TEMPERATURES

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I dedicated with love and gratitude

to my father and mother for being

with me till the very end of my thesis completion.

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Sincerely,
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ABSTRACT

Concrete filled hollow steel section columns exhibit various advantages over other materials for similar applications. These include improvement in the structural behaviour with high load bearing capacity for smaller cross-section, better appearance, rapid construction, and high fire resistance without external protection. The study of the thermal-structural behaviour of concrete filled hollow section columns has seen a gradual transition to numerical simulations over an expensive and time consuming physical tests. At present, most of the numerical tools developed in Malaysia for predicting the behaviour of structure in fire is carried out using finite difference method which can be tedious, complicated and very sensitive to numerical errors. Thus, a three-dimensional finite element model, ABAQUS, is proposed to study thermal-structural behaviour of axially loaded concrete filled hollow steel section slender columns for circular and square cross-sections at elevated temperatures. The outer diameter of the circular columns ranged from 141.3 mm to 478 mm and the steel thickness varied from 4.78 mm to 12.79 mm. The outside width of the square columns ranged from 152.4 mm to 350 mm, while the thickness of the steel wall varied from 5.3 mm to 7.7 mm. The proposed numerical models are also ranged based on types of concrete (plain and bar-reinforced concrete), steel yield strength (284 MPa to 350 MPa), concrete compressive strength (18.7 MPa to 58.3 MPa), and thickness of external protection (7 mm to 17 mm). The parameters input used in the model are results of an extensive sensitivity analysis. The accuracy of the proposed numerical model was verified against 21 experimental results and 12 existing models carried out by other researchers as well as with the predictions of the Eurocode 4 simplified calculation model. The verified model was used for a series of parametric studies on the effect of various factors affecting the fire resistance of the columns. The proposed numerical model has proved to produce a better estimation of the fire resistance of the concrete filled hollow steel section columns than the Eurocode 4 simplified model when compared with the fire tests. Based on the analysis and comparison of typical parameters, the effect of sectional shapes, concrete types and thickness of external protection on temperature distribution and structural fire behaviour of the columns are analysed. The result shows that concrete filled hollow section column with circular cross-section has higher fire resistance than square sections.

ABSTRAK

Tiang keluli berongga berisi konkrit mempunyai pelbagai kelebihan berbanding dengan material yang lain untuk aplikasi yang sama. Ini termasuk peningkatan dalam tingkah laku struktur dengan beban galas berkapasiti tinggi untuk keratan rentas yang lebih kecil, penampilan yang lebih baik, kaedah pembinaan yang pesat, dan ketahanan api yang tinggi tanpa perlindungan luaran. Kajian mengenai tiang keluli berongga berisi konkrit telah menunjukkan peralihan daripada ujian fizikal yang mahal dan memakan masa kepada simulasi berangka. Pada masa ini, kebanyakkan alat berangka yang dibangunkan di Malaysia untuk meramal kelakuan struktur dalam api dijalankan dengan menggunakan kaedah perbezaan terhingga yang mana kaedah tersebut adalah lambat, rumit dan sangat sensitif kepada kesilapan berangka. Oleh itu, model unsur terhingga tiga dimensi yang dibangunkan dengan ABAQUS dicadangkan untuk mengkaji tingkah laku struktur dan haba tiang keluli langsing berongga berisi konkrit pada suhu tinggi untuk keratan rentas bulat dan persegi. Diameter luar untuk tiang keluli berongga bulat adalah di antara 141.3 mm hingga 478 mm dan ketebalan dinding keluli adalah di antara 4.78 mm hingga 12.79 mm. Manakala lebar tiang keluli berongga persegi adalah di antara 152.4 mm hingga 350 mm, dan ketebalan dinding keluli berubah daripada 5.3 mm hingga 7.7 mm. Model berangka yang dicadangkan juga berkisar kepada jenis konkrit (konkrit biasa dan konkrit yang diperkuatkan dengan bar), kekuatan alah keluli (284 MPa hingga 350 MPa), kekuatan mampatan konkrit (18.7 MPa hingga 58.3 MPa), dan ketebalan perlindungan luaran (7 mm hingga 17 mm). Nilai yang digunakan di dalam model ini adalah hasil daripada analisis sensitiviti yang luas. Ketepatan model berangka yang dicadangkan disahkan dengan 21 keputusan eksperimen dan 12 model berangka yang dijalankan oleh penyelidik lain dan juga ramalam model pengiraan Eurocode 4 yang dipermudahkan. Model yang disahkan digunakan untuk menjalankan satu siri kajian parametrik mengenai kesan faktor-faktor yang mempengaruhi ketahan api tiang. Model berangka yang dicadangkan telah terbukti menghasilkan anggaran yang lebih baik daripada model pengiran Eurocode 4 yang dipermudahkan apabila dibandingkan dengan keputusan ujian kebakaran. Berdasarkan analisa dan perbandingan parameter tipikal, kesan bentuk keratan, jenis konkrit dan ketebalan perlindungan luaran terhadap penganggihan suhu dan tingkah laku kebakaran struktuk kebakaran tiang dibincangkan. Hasil kajian menunjukkan tiang keluli berongga berisi konkrit yang berbentuk bulat mempunyai ketahanan api yang lebih tinggi daripada yang berbentuk persegi.

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LIST OF ABBREVIATION

AFNOR	-	Association Française de Normalisation
AIDICO	-	Instituto Technologico de la Canstruccion
AIJ	-	Architectural Institute of Japan
AISC	-	American Institute of Steel Construction
ASTM	-	American Society for Testing and Materials
CFD	-	Computational fluid dynamics
CFT	-	Concrete-filled tube
CFHSS	-	Concrete-filled hollow steel section
CHS	-	Circular hollow section
EC1	-	Eurocode 1
EC2	-	Eurocode 2
EC3	-	Eurocode 3
EC4	-	Eurocode 4
FC	-	Steel fibres reinforced concrete
FCHSC	-	Fibre-reinforced high strength concrete
FDM	-	Finite difference method
FDNY	-	Fire Department of the City of New York
FDS	-	Fire Dynamic Simulator
FEA	-	Finite element analysis
FEMA	-	Federal Emergency Management Agency
HSC	-	High strength concrete
HSS	-	Hollow steel section
ISO	-	International Organization for Standardization
JIS	-	Japanese Industrial Standards
NIST	-	National Institute of Standards and Technology
NRCC	-	National Research Council of Canada
PC	-	Plain concrete

RC	-	Reinforced concrete
SCC	-	Self-compacting concrete
CHS	-	Circular hollow section
SHS	-	Square hollow section
SEG	-	Shenzhen Electronics Group
3D	-	Three dimensional

LIST OF SYMBOLS

θ_g	-	Compartment gas temperature
t	-	Fire exposure time
ρ	-	Density
c	-	Specific heat
T	-	Temperature
q	-	Heat flux vector per unit time
Q	-	Internal heat generation per unit volume
k	-	Thermal conductivity of the material
h_{net}	-	Net heat flux
h_{conv}	-	Convection heat flux
h_{rad}	-	Radiation heat flux
h_c	-	Convective heat transfer coefficient
h_g	-	Conductance for the gap consisting of air
T_g	-	Temperature of the gas near the column
T_s	-	Temperature of the column surface
φ	-	Configuration factor
ε_m	-	Emissivity of the column member
ε_f	-	Emissivity of the fire
σ	-	Stefan-Boltzman
T_r	-	Temperature of effective radiation
T_m	-	Surface temperature of the column membe
D	-	Diameter
B	-	Width
L	-	Length

λ_a	-	Thermal conductivity of steel
c_a	-	Specific heat capacity of steel
θ_a	-	Temperature of steel
λ_c	-	Thermal conductivity of concrete
c_p	-	Specific heat capacity of concrete
θ_c	-	Temperature of concrete
ε	-	Strain
σ	-	Stress
E	-	Elastic modulus
$f_{y,\theta}$	-	Effective yield strength;
$f_{p,\theta}$	-	Proportional limit;
$E_{a,\theta}$	-	Slope of the linear elastic range;
$\varepsilon_{p,\theta}$	-	Strain at the proportional limit;
$\varepsilon_{y,\theta}$	-	Yield strain;
$\varepsilon_{t,\theta}$	-	Limiting strain for yield strength;
$\varepsilon_{u,\theta}$	-	Ultimate strain.
σ_{true}	-	True stress
ε_{true}	-	True strain
$\sigma_{no\min al}$	-	Normal stress
$\varepsilon_{no\min al}$	-	Normal strain
$\varepsilon_{plastic}$	-	Plastic strain
$\varepsilon_{elastic}$	-	Elastic strain
f_{c}	-	Cylinder strength of concrete at temperature
f_{co}	-	Cylinder strength of concrete at room temperature
ε_{max}	-	Maximum strain of concrete
v	-	Poisson’s ratio
β	-	Angle of friction
K	-	The ratio of flow stress in tri-axial tension to compression
ε_c	-	Total strain;

ε_{th}	-	Free thermal strain
ε_{σ}	-	Instantaneous stress-related strain
ε_{cr}	-	Classical creep strain
ε_{tr}	-	Transient strain (stress and temperature dependent)
ε_{th}	-	Free thermal strain
θ_{c0}	-	The initial temperature of concrete
θ_{m}	-	Surface temperature of the member
θ_{r}	-	Effective radiation temperature of the fire environment
$A_{contact}$	-	Nominal contact area
ΔT	-	Additional temperature drop due to the presence of imperfect contact
$N_{fi,Rd}(\varepsilon)$	-	Design value of the fire resistance of the column in axial compression
$N_{fi,cr}(\varepsilon)$	-	Euler buckling load of the composite column at elevated temperature
$N_{fi,pl,Rd}(\varepsilon)$	-	Design value of the plastic resistance to axial compression of the cross-section subjected to fire
λ	-	Relative slenderness at room temperature
l_{θ}	-	Buckling length of column in fire
R	-	Standard fire resistance
$(EI)_{fi,eff}$	-	Cross-section effective flexural stiffness
A	-	Area of cross-section
f_{θ}	-	Design strength of material at a certain temperature
γ_{M}	-	Partial safety factor in fire condition
a	-	Steel
c	-	Concrete
s	-	Reinforcing steel
$E_{\sec,\theta}$	-	Secant modulus at a certain temperature
$f_{c,\theta}$	-	Design compression strength of concrete

$\varepsilon_{c1,\theta}$	-	Corresponding strain at peak stress
$\bar{\lambda}_{\theta}$	-	Relative slenderness of the column at elevated temperature
$N_{fi,pl,R}$	-	The value of $N_{fi,pl,Rd}$ when the material partial safety factors are taken as unity
χ	-	Reduction coefficient from buckling curve "c"
A_m / V	-	Section factor
$\theta_{a,eq}$	-	Equivalent temperature for the steel tube
λ_p	-	Thermal conductivity of the fire protection system
A_p / V	-	Section factor for steel members insulated by fire protection material
A_p	-	The appropriate area of fire protection material per unit length of the material
V	-	Volume of the member per unit length
d_p	-	Thickness of the fire protection material
ρ_a	-	Unit mass of steel
$\theta_{g,t}$	-	Ambient gas temperature at time t
$\theta_{a,t}$	-	Steel temperature at time t
c_p	-	Temperature independent specific heat of the fire protection material
ρ_p	-	Unit mass of the fire protection material
$\theta_{c,eq}$	-	Equivalent temperature of the concrete core
$\varphi_{a,\theta}$	-	Correction factor
B	-	Factor of fire resistance time
D_s	-	Width of steel tube

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CHAPTER 1

INTRODUCTION

1.1 General

Over the past few decades, fire research in steel and steel-concrete composite structures has evolved tremendously from experimental measurements to computational methods (Cote, 2003). The research is driven by the need for a better understanding in the fire behaviour of building structures so that structural and economical design for fire safety can be improved. One way of measuring the fire performance of a structure is by its ability to withstand the exposure of a standard fire for a period of time without losing its structural stability and integrity. This is referred to as "*fire resistance*" by most building codes and material standards (Franssen & Kodur, 2009). Therefore, it is extremely important to design a building considering the fire resistances of the various building elements used in the assembly.

In construction industry, structural redundancy is the essence of high-rise building structures as it allows for the loss of one primary structural member without collapsing the entire structure. According to John Kenlon, the FDNY Chief of Department in the early 1900s, the fire induced collapse of a steel framed structure is mainly caused by the failure in columns (Hill, 2012). Moreover, the evidence from the World Trade Centre collapse indicated that the weakened columns as the impact result of aircraft fires attributed to the progressive collapse of the towers (FEMA, 2002, 2005). The brief background brought most attention to the researcher hence launched the study in the performance of concrete-filled hollow steel section (CFHSS) columns in fire condition.

1.1.1 Advantages and Disadvantages of CFHSS Columns

Fire has destructive effect on structures as its strength and stiffness deteriorate at high temperature, consequently affecting the stability of a building. These effects dependent on the thermal and mechanical properties of the materials comprising the structural elements. Among all the elements, columns appears to be the most critical components as its failure could lead to partial or complete collapse of a building.

The basic forms of CFHSS columns can be introduced in three forms – circular, square and rectangular, as illustrated in Figure 1.1. They have been widely accepted by structural engineers and designers for high rise construction due to the benefits of combining steel and concrete. The marriage of these two building materials, viz concrete and steel, is practically intended complement the deficiency of both structural steel columns and conventional reinforced concrete. For instance, the composite column is designed in such a way that concrete is utilized to resist compression while steel in tension. Overall, the advantages of CFHSS columns can be viewed in four perspectives – structural, architectural, construction, and economical, which can be explained as follows: (Bergmann, Matsui, & Meinsma, 1995; Twilt, Hass, Klingsch, Edwards, & Dutta, 1996; Wardenier, 2001; Zhao, Han, & Lu, 2010).

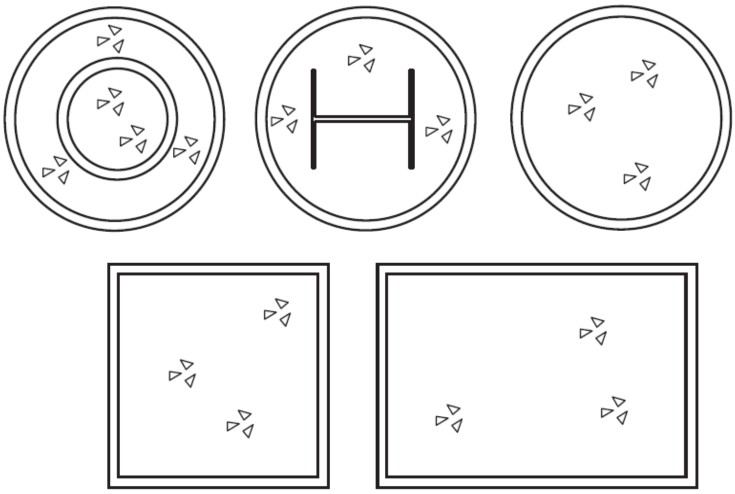

Figure 1.1 Types of CFHSS columns (Ranzi, Leoni, & Zandonini, 2013)

From a structural aspect, the concrete core promotes a higher rigidity and load bearing capacity to the tubular section where high load is able to sustain by the slender columns without increasing the size of the cross-section. Moreover, the concrete infill also restrains any possible inward local buckling of the steel section thus increases the flexural stiffness and ultimate strength of the column in a frame. On the other hand, the steel shell provides confinement to the concrete core to prevent excessive spalling under loading condition (Jacobs & Goverdhan, 2008; Y.C. Wang, 2002). It also increases strength and ductility of the column as well as its energy-absorption capacity which makes it feasible for seismic design.

In architectural design standpoint, because extra load capacity is obtained for CFHSS columns for the same dimension column steel and column reinforced concrete section, larger usable floor space within a building is achieved due to the reduction in the required size of the cross-section. This will enhance the use of CFHSS especially in the lower storeys of tall buildings and parking garages where the columns experiences higher forces than those in storeys above. In addition, the exposed hollow sections are preferred by most architects because of aesthetic value and robustness of these columns in structural design.

In terms of construction, formwork for CFHSS column is not required because steel hollow sections provides an integral support for concrete as it hardened subsequently. This method of construction allows for much efficient in construction process as the erection of structural steel components into a steel frame can be done before or simultaneously with the casting of concrete. Also, with the inclusion of concrete in the void area of steel section, smaller columns and rapid construction are achieved, hence leads to the reduction in manpower and time of the project.

Even though the cost production of hollow sections is higher than those for conventional open sections, they offer economic benefits in other areas including those mentioned in the previous paragraphs. In addition, its lower surface area reduces painting and corrosion protection costs, thus reduces the total expenditure on construction (Wardenier, 2001).

Although CFHSS columns possess a numerous advantages as mentioned above, however, short of knowledge about the construction methods and fire dynamic design limits its application (Capilla, 2012). Although few design procedures and calculation methods have been proposed, these methods show inaccurate results at an elevated temperature (Capilla, 2012). Therefore, more studies are needed to understand the construction method and to develop design procedures for CFHSS columns.

1.1.2 Practical Applications of CFHSS Columns in Buildings

As mentioned above, CFHSS columns have high load bearing capacity and improved fire performance as compared to tubular steel sections. These advantages have led to its wide use in practical applications. Among many applications, most prominent are the high-rise buildings and bridges. Most of these bridges and buildings are located in China, Japan, USA, England and Canada as described in various publications (Capilla, 2012; Wardenier, 2001; Zhao et al., 2010). SEG Plaza in Shenzen and Wuhan international securities buildings are the examples of CFHSS column based high-rise buildings in China (Zhao et al., 2010). SEG plaza, as shown in Figure 1.2, is 64-storey office building which employs circular shaped CFHSS columns. Further, more than 100 bridges have been built in CHINA using CFHSS columns (Zhao et al., 2010). Ikeda and Ohmiya (Ikeda & Ohmiya, 2009) reported various design applications of CFHSS columns without external fire protection in buildings of Japan during 1993 to 2004. Some of the examples are Mitsui Soko Hokozaki Building (Tokyo) and ENICOM computer centre (Tokyo). Kodur and Mackinnon (2000) presented a review of various applications of CFHSS columns in USA and Canada. Museum of flight at King Country Airport (Washington, USA) and St. Thomas elementary school (Ontario, Canada) used CFT columns to achieve high fire resistance. Some examples of used of CFHSS column buildings of London (United Kingdom) include fleet palace house and Peckham library (Hicks & Newman, 2002) as illustrated in Figure 1.3. Further, other examples of CFHSS columns in building design include Technocent building (Finland), Amsterdam mees lease building (Netherlands), and Riverside office building (Australia) (Twilt et al., 1996).

Figure 1.2 SEG Plaza (Shenzen, China)

Figure 1.3 Fleet Place House (London, UK)

1.2 Problem Statements

Structural fire design has seen a gradual transformation from prescriptive-based to performance-based approach in many places around the world including the US, UK, Spain, Australia and China (Espinos, Romero, & Hospitaler, 2010; Hong & Varma, 2009; Lu, Zhao, & Han, 2009; Zha, 2003). The prescriptive-based approach relies heavily on the results interpretation of the attained from standard fire tests. Thus, this approach is restricted to architectural and aesthetic requirements. Meanwhile, the performance-based approach requires knowledge in the principle of fire science, heat transfer, and structural mechanics to develop a numerical tool to identify the performance of a structure on fire occurrence. In that way, a through understanding in the behaviour of CFHSS columns under extreme loading and fire conditions can be attained, hence fire safety can be improved without jeopardising the design flexibility and cost for fire protection.

At present, most of the numerical tools developed in Malaysia are mathematical-based which is carried out using finite difference method (FDM) (Abd El-Ghani, 1998; Alham, 2000; Mian, 1998; Shehata, 2001). It is extremely tedious and complicated, not to mention that the results are prone to inaccuracies because of various assumptions were made to simplify the problems, for instance the mechanical and thermal interactions between steel and concrete (Ghojel, 2004). Therefore, numerical modelling is seen as an alternative approach to stimulate a realistic behaviour of CFHSS columns at elevated temperature (Espinos et al., 2010). In addition, extensive parametric studies can be carried out to explore further behaviour of CFHSS columns in order to support the fire design for columns without the execution of physical tests.

1.3 Aim and Objectives

The principal aim of this research is to develop new and efficient 3D numerical models for predicting the thermal and structural behaviour of CFHSS columns at elevated temperatures. In order to achieve this aim, the objectives are outlined as follows:

1. To develop a 3D thermal-structural model for CFHSS circular and square columns subjected to the standard fire test.

2. To verify the 3D thermal-structural model by comparison with existing experimental and numerical results.

3. To compare the accuracy of the numerical predictions with those obtained from Eurocode 4 simplified calculation model against experimental results.

4. To undertake parametric studies to examine the effect of changing important parameters on the behaviour of CFHSS columns under standard fire condition

1.4 Research Methodology

To achieve the aforementioned objectives, the following brief research methodologies are identified herein.

1. Nonlinear finite element analysis (FEA) model by ABAQUS was carried out to calculate the temperature distribution within the cross-section and the temperature gradient along the member length.

2. Nonlinear FEA model by ABAQUS was adopted to determine the fire resistance and displacement of CFHSS column at elevated temperature.

3. Validation of the FEA model with the available test results has been done. The list of test results for the validation of FEA model are shown in Table 1.1 and 1.2, for circular and square columns, respectively. The outer diameter of the circular columns ranged from 141.3 mm to 478 mm and the steel thickness varied from 4.78 mm to 12.79 mm. The outside width of the square columns ranged from 152.4 mm to 350 mm, while the thickness of the steel wall varied from 5.3 mm to 7.7 mm. The proposed numerical models are also ranged to types of concrete (plain and bar-reinforced concrete), steel yield strength (284 MPa to 350 MPa), concrete compressive strength (18.7 MPa to 58.3 MPa), and thickness of external protection (7 mm to 17 mm).

4. Sensitivity study to eliminate the uncertainties associated with the output of the numerical model. The input parameter studies include the Poisson's ratio of concrete, thermal conductance at the steel-concrete interface, friction coefficient at steel-concrete interface, imperfection buckling of columns, concrete plasticity model and material mechanical models at elevated temperature.

5. Comparison of the numerical model for CFHSS columns with Eurocode 4 simplified calculation model.

6. Parametric studies to explore the effect of cross-sectional size, concrete types and the thickness of external fire protection on the fire resistance of steel tube with circular and square cross-section column filled with concrete.

7. Conclusion and recommendations based on the analysis and numerical results are drawn with suggestions for further work.

1.5 Scope and Limitation

The scope of this research is limited to CFHSS column of circular and square shape, filled with normal strength concrete and subjected to concentric loads. The research is also limited to plain and bar-reinforced concrete fillings, steel yield strength (284 MPa to 350 MPa) and the concrete compressive strength (18.7 MPa to 58.3 MPa).

This work will focus primarily on slender columns with length of 3810 mm for predicting the behaviour of columns at elevated temperature. For circular sections, the outer diameter is ranged from 141.3 mm to 478 mm and the steel thickness varied from 4.78 mm to 12.79 mm. For square sections, the outer width is ranged from 152.4 mm to 350 mm and the thickness of the steel wall is varied from 5.3 mm to 7.7 mm.

For the extension of this work, external fire protection with varying thickness between 7 mm to 17 mm is also included in the validation process of the model. Moreover, this work will focus mainly on axial loaded columns subjected to standard fire of either ASTM E-119 (ASTM, 1990) or ISO-834(ISO, 1980).

1.6 Significance of Research

The research findings from this project provide significant contribution to the understanding of the behaviour of CFHSS columns subjected to fire and axial loads. The proposed 3D numerical models for predicting the thermal and mechanical response of the column provide structural design with an advanced analysis and design tools that can be used in fire design. In addition, the incorporation of the temperature dependent formulations factors gives more realistic representation of the behaviour of axially loaded CFHSS columns at elevated temperature.

1.7 Thesis Layout

The contents of this thesis are divided into 6 chapters. Chapter 1 is the introduction part which highlighted the background of the research work, the objectives and significance of the research work. Chapter 2 presents an extensive literature reviews on experimental investigations, analytical approaches, numerical methods, and calculation models on the behaviour of CFHSS columns at elevated temperature.

Chapter 3 discusses in detail the research methodology adopted in the study. The first task focuses on developing the proposed numerical model on ABAQUS for investigating the behaviour of CFHSS column at elevated temperature. The second task focuses on developing the Eurocode 4 simplified calculation model for predicting the fire behaviour of the CFHSS column.

Chapter 4 validates the proposed numerical model with a series of fire tests by various researchers. This chapter also compares the results obtained from the proposed numerical model against those obtained from the EC4 simplified calculation model and numerical models by previous researchers.

Chapter 5 conducts parametric studies to explore the effect of cross-sectional size, concrete types and thickness of external fire protection on the fire resistance of CFHSS column by using the numerical model that was validated in the previous chapter. The study for investigating the effect of cross-sectional size consists of three cases which include: (i) equal section strength at ambient temperature, (ii) equal steel cross-sectional area, and (iii) equal concrete-cross-sectional area. Meanwhile, there are three concrete types investigated which include (i) bare, (jj) plain concrete, and (iii) reinforced concrete. Lastly, the study on effect of thickness of the protection ranged from 0 mm, 10 mm and 25 mm.

Chapter 6 presents the summary of the entire work, conclusions as well as recommendations for future work.

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