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Shear resistance of concrete filled steel tubular columns at elevated temperature

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ZADÁNÍ DIPLOMOVÉ PRÁCE

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Název diplomové práce anglicky: Shear resistance of steel tubular columns filled with fibre reinforced concrete at elevated temperature.

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State of the Art
FEA model of pure shear resistance of fiber reinforced concrete (FRC) at ambient temperature.
Validation of the model against experimental result at ambient temperature.
Numerical Experiment for the shear properties of FRC at elevated temperature.
Statical studies for the shear and moment distribution and their interaction.
Failure mode study considering the eccentiricity and other relevant parameters.
Summary

Seznam doporučené literatury:
Performance-Based Fire Engineering of Structures,Yong Wang, Ian Brugess, František Wald & Martin Gillie
Guidelines for Fire Resistant Design of Concrete-Filled Steel HSS Columns - State-of-the-Art and Research Needs. Kodur, V. Steel Structures 7 (2007) 173-182.

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III. PŘEVZETÍ ZADÁNÍ

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DECLARATION

I declare that this thesis has been composed solely by myself and that it has not been submitted, in whole or in part, in any previous application for a degree or processional qualification either in Czech Technical University or elsewhere. Except where states otherwise by reference or acknowledgment, the work presented is my own.

Prague, 7 of January 2018 Tesfamariam Arha

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ABSTRACT

Combining different materials in a single structural member to take advantage of the good qualities that they both have separately has always been a recognized strategy in building industry. Concrete filled steel tubular columns (CFST) are a type of composite columns in which the combined action of steel and concrete leads to an exceptional structural behavior. In this case, the compressive strength of the element increases due to the passive confinement that the steel tube generates on the concrete core. Simultaneously, the local buckling of the steel tube is improved due to the support of the concrete core which prevents it from suffering this phenomenon inwards. In concrete filled tubular columns (CFST) the combined action of steel and concrete results in many positive attributes at ambient temperature: high load-bearing capacity with smaller cross-section size, aesthetics, high stiffness and ductility and reduced construction cost.

In recent years, the use of concrete filled tubular columns in construction industry, especially in high-rise buildings, has increased not only because of these positive characteristics at room temperature, but also for their inherent high fire resistance. The ambient temperature behavior of CFST columns has been deeply studied and, in turn, the investigations dealing with their fire behavior have increased.

In this thesis the shear resistance of CFST columns at elevated temperature is investigated. There is little or no data in the literature regarding the shear resistance of CFST columns.

At elevated temperatures due to the difference of thermal expansion, steel and concrete separated from each other. The steel suffers local buckling and most of the load is carried by the concrete. Therefore, in this dissertation a simplification is made by considering only the concrete part which contributes to the shear resistance of the column.

Pure shear resistance of fiber reinforced concrete at elevated temperature is investigated and studied in detail. The present study explores an experimental and numerical study on the pure shear resistance of fiber reinforced concrete. A numerical model is developed and validated against the experimental result. Using the validated model a parametric study on the shear resistance of fiber reinforced concrete at elevated temperatures was performed.

In addition, a numerical studies were done in order to identify the changes in the mechanical properties of fiber reinforced concrete due to variation in the characteristic size of the material. Besides to that a parametric study was also done numerically on the model of a fiber reinforced concrete column in order to further understand the behavior of fiber reinforced concrete column under combined action of loadings.

Thus, by concluding with all information and results achieved with this study, it is intended to enrich the existing limited knowledge on shear resistance of CFST columns at elevated temperatures.

Keywords: concrete filled steel tubes (CFST), circular hollow section (CHS), steel fiber reinforced concrete (SFRC), elevated temperature, and shear resistance.

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1 INTRODUCTION

This chapter introduces the structural technology of concrete filled steel tubular columns and their fire resistance characteristics. The benefits of using these composite columns over the other alternatives are also described. The three problems which constitute the whole fire analysis of a CFST column are explained: the fire dynamics, the thermal and the structural analysis.

1.1. Background

Combining different materials in a single structural member to take advantage of the good qualities that they both have separately has always been a recognized strategy in building industry. Concrete filled steel tubular columns (CFST) are a type of composite columns in which the combined action of steel and concrete leads to an exceptional structural behavior. In this case, the compressive strength of the element increases due to the passive confinement that the steel tube generates on the concrete core. Simultaneously, the local buckling of the steel tube is improved due to the support of the concrete core which prevents it from suffering this phenomenon inwards.

Structural hollow sections (SHS) are the most efficient steel sections in resisting compression loads. Generating a new composite section by filling the hollow tube with concrete allows not only retaining all the advantageous features of the empty section but also developing new excellent properties. This fact reveals the synergy existing when these two elements work in conjunction. Figure 1.1 shows the behavior of a steel tubular column, a reinforced concrete (RC) stub column and a concrete filled steel tubular stub column without reinforcing bars. As it can be observed, the summation of the steel tube and the RC columns ultimate strength is less than that achieved by the CFST column Han et al. 2014.

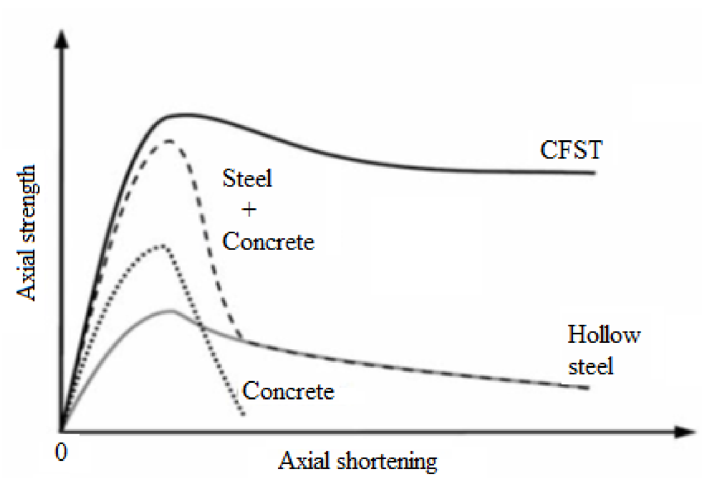


Figure 1-1 Axial compressive behavior of stub columns Han et al. 2014.

Concrete filled steel tubular (CFST) columns have many positive attributes at ambient temperature for building industry: high load-bearing capacity with smaller cross-section size, attractive appearance, high stiffness and ductility, high seismic resistance and reduced construction cost and time since no formwork is necessary.

In recent years, the use of CFST columns in construction industry, especially in high-rise buildings, has increased not only because of these positive characteristics at room temperature, but also for their inherent high fire resistance. The joined action of the steel tube and the concrete core leads to an excellent fire resistance behavior: the concrete core retards the heating of the steel tube, while, at the same time, the steel tube protects the concrete core from direct fire exposure, thus delaying the integrity loss of the concrete, which, furthermore, degrades slower than steel under fire Twilt et al. 1996.

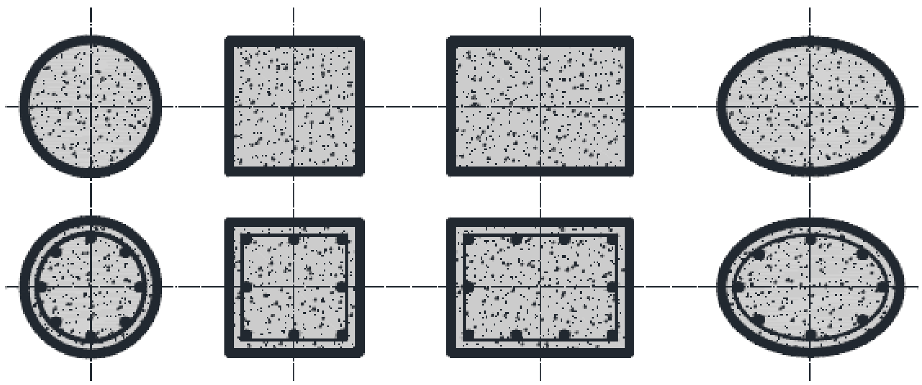


Figure 1-2 Possible shapes of CFST cross-sections Han et al. 2014

Among the most commonly used shapes of CFST columns, it can be found circular and rectangular cross-sections Figure 1.2, although new shapes, such as elliptical profiles, are appearing. A wide variety of concrete infills can be used such as plain concrete, bar-reinforced concrete or fiber reinforced concrete. As mentioned above, the combination of these elements permits the design of columns with reduced cross-section. However, when free spaces and higher net usable surfaces are desirable, the cross-section of the CFST column can be even smaller if high strength materials are employed, solution which is hugely applied by high-rise buildings designers.

At elevated temperatures in case of fire, the behavior of a structure is much more complicated comparing at the ambient temperature. The structural behavior is highly nonlinear and inelastic due to the changes in material properties and thermal movement. So far, it has not been possible to develop an analytical methods to investigate the behavior and properties of CFST columns in fire with sufficiency accuracy and numerical simulations are necessary. In general, the calculation of the fire resistance of a column involves calculation of the temperatures of the fire to which the column is exposed, the temperature in the column and its deformations and strength during exposure to the fire. In fact, the analytical methods which can be found in the literature Lie 1984 provide only approximations to the solutions but not a realistic representation of the fire response of CFST columns.

Until now, a large number of numerical models have been developed worldwide Zha 2003, Ding & Wang 2008,which have helped to gain insight into the fire behavior of this type of composite columns. However, a number of key factors which are sometimes neglected or treated on a simplified manner need to be taken into account in the numerical models in order to obtain a more realistic representation of the fire behavior of these types of composite columns.

The interest in the use of simple design rules for concrete-filled steel tubular columns has grown in the last decades, due to the increased usage of this structural typology. Nevertheless, only a limited number of methods are available to designers for evaluating the fire resistance of this type of composite columns, which are a result of the numerical and experimental investigations carried out by the main research groups working in this field Han et al. 2003, Kodur 1999.

In Europe, the current design rules for CFST columns at elevated temperatures are under review as it has been proved to be unsafe and besides presents some limitations according to Romero et al 2011, and Renaud et al. 2004. Therefore, further investigation on the technology and design of CFST columns is needed to promote their usage and make designers more familiar with the available calculation methods.

1.2. Fire behavior of CFST columns

The use of concrete filling offers a practical alternative for providing the required fire resistance in steel hollow structural section columns without the need of additional protection Kodur and Lie 1995 and Twilt et al.1996.This is due to the heat sink effect produced in the composite section because of the low thermal conductivity of concrete and the mechanical contribution of the concrete core, which helps to support the applied load and also prevents the inward local buckling of the steel.

The increment in the fire resistance rate (FRR) can be magnified with the use of internal reinforcement. The fire resistance reached by bar-reinforced concrete (RC) filled tubular columns is higher than that achieved by steel tube columns filled with fiber reinforced concrete (FRC), which, at the same time, is higher than the FRR shown by plain concrete filled hollow steel section columns Kodur 2007.

Figure 1.1 illustrates the comparison of the FRR achieved by the three types of concrete filling for three columns with different dimensions and shapes. As it can be observed, the FRR reached can have a difference of more than 120 minutes between columns depending on their size, section and type of filling.

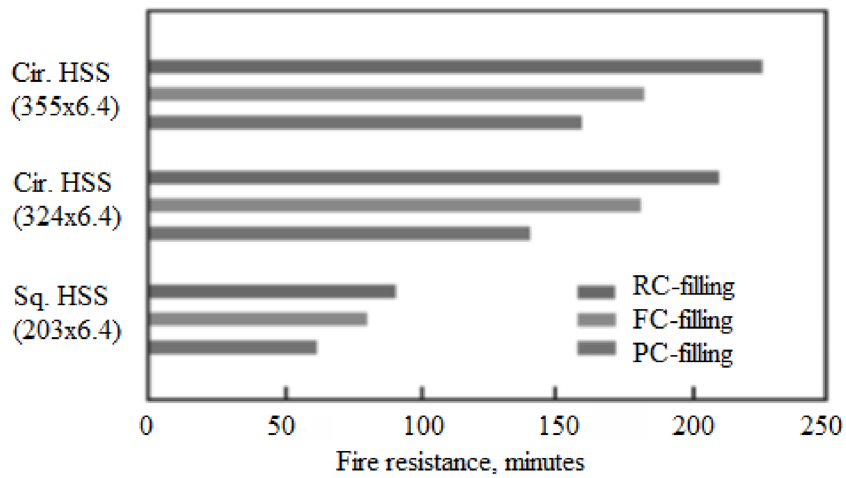


Figure 1-3 Effect of using different types of concrete infill on the fire resistance of CFST columns Kodur 2007.

Although filling the hollow steel section column with plain concrete (PC) without any type of steel reinforcement offers the most economical combination from the perspective of fire behavior, in some cases, especially with large column dimensions, PC filled columns fail at a relatively low applied loads during fire exposure. Excessive local stresses in the concrete core due to the loss of compressive strength with temperature and also the early cracking originated by the strength reduction in the steel tube exposed to fire can be the sources of this premature failure Lie and Kodur 1996.

The existence of steel bars in a reinforced concrete CFST column not only improves the load bearing capacity of the concrete core but also prevents concrete from crack propagation and sudden strength loss. Nevertheless, it implies the additional cost of steel bars and their installation in the CFST column Lie and Kodur 1996.

The use of fiber reinforced concrete in CFST columns increases the load bearing capacity of the column to a certain degree and provides better fire behavior than plain concrete infill. In this case, the supplement in the cost comes only from the steel fibers Kodur 2007.

When the column is exposed to fire, the steel carries most of the load during the early stages because the steel section expands more rapidly than the concrete core. The heat flux is gradually transferred from the steel tube wall to the concrete infill, where the temperature increases relatively slow due to the low thermal properties of concrete. As temperature increases, the steel section gradually yields as its strength decreases, and the column rapidly contracts at some point between 20 and 30 minutes after exposure to fire. At this stage, the concrete filling starts carrying more and more of the load. The strength of the concrete decreases with time and ultimately, when the column can no longer support the load, either buckles or fails in compression. The time at which the column fails determines its fire-resistance rating. This evolution can be observed in Figure 1.2 in terms of the axial displacement measured at the top of the column.

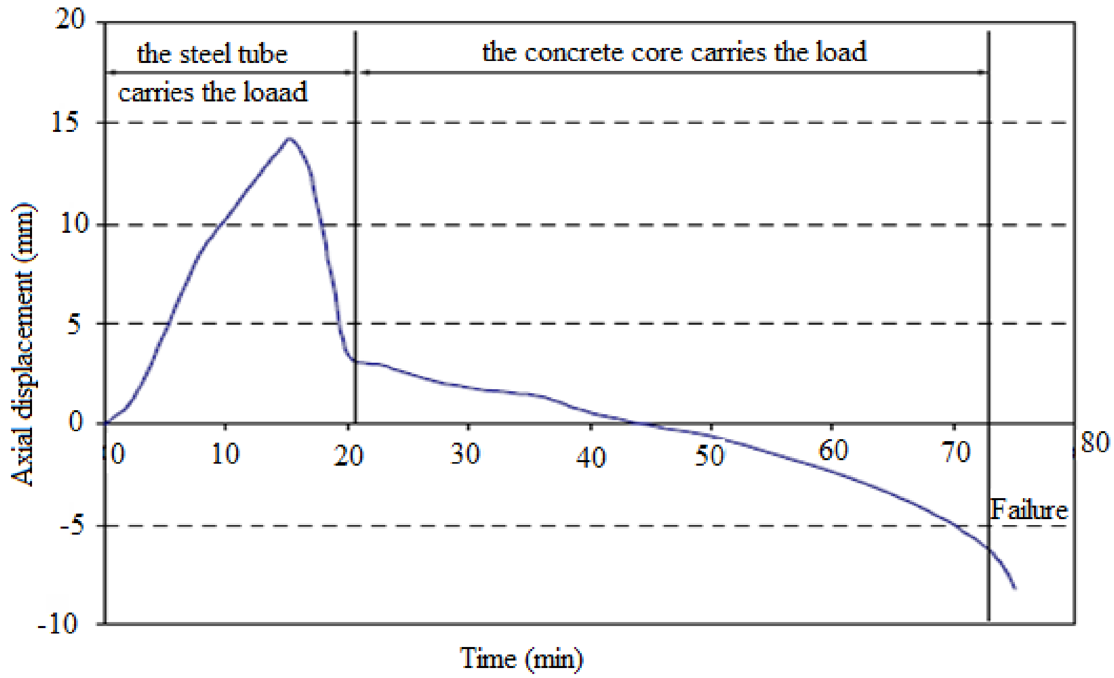


Figure 1-4 Axial displacement versus time curve of a typical behavior of CFST column.

At elevated temperatures, the moisture present in the concrete core is released in the form of water vapor, and in order to avoid the development of excess internal pore pressure, it is necessary to facilitate its release Twilt et al. 1996, for this reason vent holes at the end of each column are recommended.

The heating behavior of CFST columns is completely different than that of empty steel hollow sections. Both concrete and steel have different thermal conductivity, thus producing high transient heating behavior charcterized by high temperature differentials across the cross section Twilt et al. 1996. Therefore, CFST columns can be designed succesfully to achieve the fire requiremnets with out external fire protection, taking advantage of the temperature differentials of the concrete core and the steel tube.

The different components of a CFST column experience different degradation rate with respect to the fire exposure time, depending on their particular location within the cross-section Twilt et al. 1996. The steel tube which is directly exposed to fire, heats up quicker and results significantly in its decrease of resistance after a short period of time. On the other hand, the internal core of concrete with low thermal conductivity keeps most of its ambient temprature strength with out much decrease. If reinforcing bars are used, they are placed close to the external surface, but still there is some cover to protect them against the direct exposure to fire, thus the degradation of their strength is gradual. Figure 1-5 shows the behavior of the different components of CFST column cross-section.

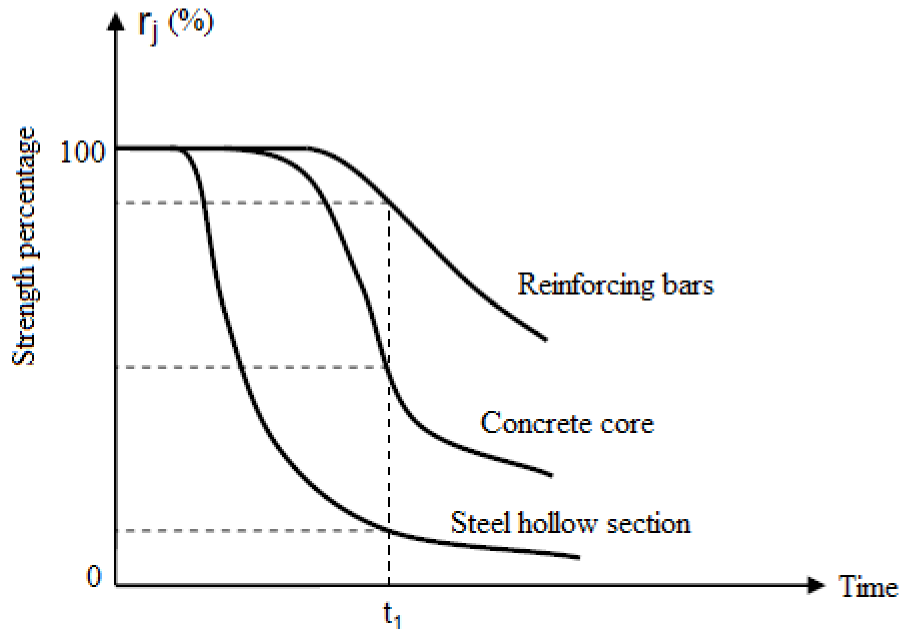


Figure 1-5 Evolution of strength of the different components of CFST column

According , Twilt et al. 1996, the load carrying capacity of CFST column at elevated temperature, is obtained by summing up the resitance of each component of the composte column which depends on the fire exposure time.

At ambient temperature most of the load is carried by the steel tube, due to its high strength and the position, but with the increase of temperature after a period of time, the steel loses most of its strenght and the load is transferred from the steel tube to the concrete which heats up gradually.

Similarly, a minimum dimension of CFST column cross-section is necessary in order to fulfill the fire requirments Twilt et al. 1996, because the strength reduction of the various components is affected by the rate of heating, which inturn depends on the dimnesion of the components. With the increase of temperature, the strength and stiffness of the materials decrease,resulting in increased deformation. Therefore,in the fire design, account must be taken of the slenderness of the column.

Studying the structural behavior of a member is complicated process since it involves many variables such as fire growth and duration, temperature distribution in structural members, interaction between structural members, changes in material properties and the effect of loads.

Therefore, in order to compute the fire resistance time of a structural member and give an accurate treatment to all these variables, it is necessary to address three problems of different nature: a fire dynamics analysis, a thermal analysis and a structural analysis. The adoption of a fire dynamics model allows the determination of the heating regime affecting the member; by means of the heat transfer model the element temperatures evolution is computed; and the structural model permits the calculation of forces and stresses to evaluate when the collapse occurs.

The next sections deal with the description of the basis of each of these problems which need to be solved to obtain the fire resistance of a CFST column.

1.2.1. Fire dynamics analysis

The temperature evolution of a compartment where the structural element is located is predicted using the fire dynamics model. The amount of energy released depends on the volume of the combustible materials and the ventilation.

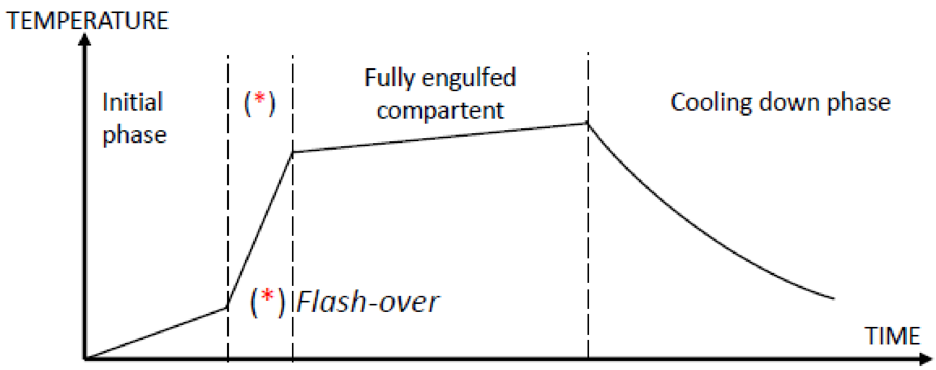


Figure 1-6 Different stages in the evolution of a fire

The evolution of fire can be divided in to 4 stages. The initial phase, flush-over phase, fully developed phase and cooling phase. The temperature rises very rapidly after the flush-over, which is the point where most of the combustible materials found in the compartment burn out spontaneously.

To evaluate the response of the structure in case of fire, the first step consists in characterizing the fire action. In practice, one can use models of different sophistication levels, depending on the applications. The action of the fire on a structure can be represented using models. The models are chosen based on either the prescriptive approach which uses the standard or nominal fire curves or the performance based approach using the natural fire.

The nominal fire curves described in EN 1991-1-2 (CEN 2002), section 3.2 are conventionally agreed curves which are used to classify or for standard representation of the resistance of the structural element. The available nominal fire curves are the nominal standard temperature-time curve (ISO 834 curve), the external fire curve and the hydrocarbon curve.

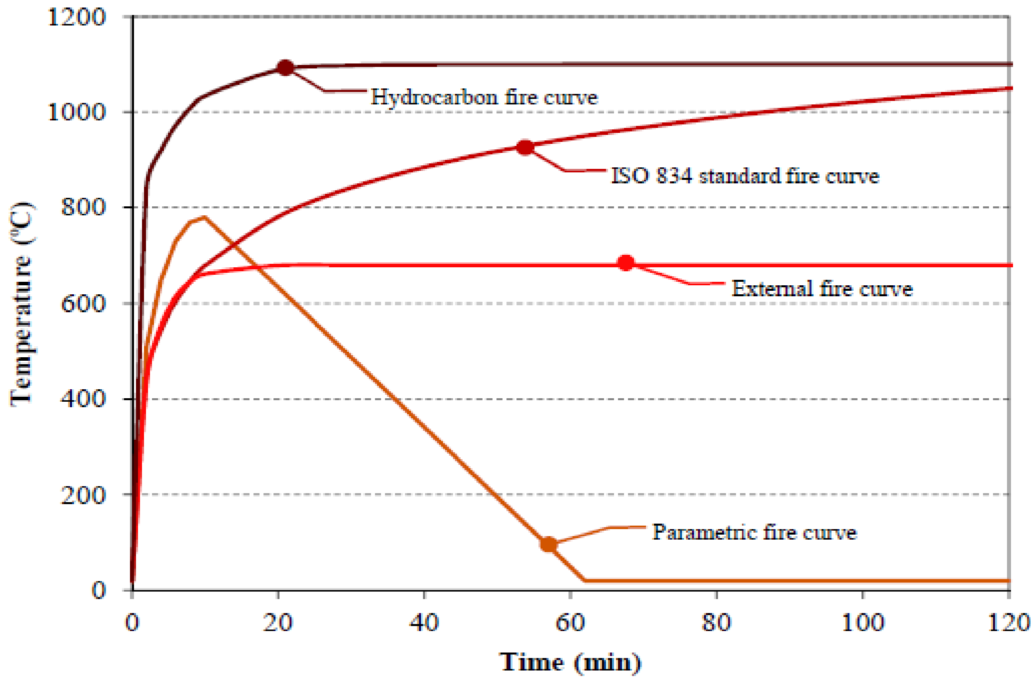


Figure 1-7 Different types of fire curve

The natural fire models are described in section 3.3, EN 1991-1-2 (CEN 2002). They include the simplified models either for localized or compartment fires which consider at least the fire load density and the ventilation conditions , and the advanced fire models which takes into account other parameters such as the gas properties, the mass exchange and the energy exchange. The latter category includes one-zone model, two-zone model or the computational fluid dynamics model (CFD).One –zone model assumes a uniform, time dependent temperature distribution in the compartment. Two-zone model considers an upper and lower layer with time dependent thickness and time dependent uniform temperature. Finally, CFD model give temperature evolution in the compartment in a completely time and space dependent manner.

Conversely, a natural fire model can be simulated in a more simplified manner by parametric fire curves. These curves assume a uniform temperature distribution in the fire compartment and have limited field of application since they consider specific physical parameters regarding the fire load, the openings characteristics and the enclosure thermal properties. Parametric fire curves usually display both growing and cooling branches which make them not practical for testing structural elements since simulating the descending part may be difficult to control by furnace systems. On the contrary, these set of curves are very useful for design and they give an improved prediction compared to that offered by nominal fire curves. Nevertheless, their application requires the election of the proper material constitutive equations which include the modeling during the cooling situation.

In many building codes, the fire resistance of a structural element is based on its performance when it is exposed to a fire pattern which follows an internationally agreed temperature-time curve, i.e.

ISO 834 standard and does not represent a realistic fire. It is characterized by a gas temperature which increases continuously along the time, but in a slow rate. It has become a standard pattern which is used in the laboratories for testing the resistance of structural elements in fire.

The standard ISO 834 curve is described in Section 3.2.1 of Euro code 1 Part 1-2 (CEN 2002) and it is given by the formulae:

$\theta_g = 20 + 345. log_{10}(8. t + 1) \quad [°C]$ …………………………..(1.1)

Where:

- θ_g is the gas temperature in fire compartment in $[°C]$
- t is the time in [min]
- A coefficient of heat transfer by convection $\alpha_c = 25\ W/m^2K$ is used

1.2.2. Heat transfer analysis

The main objective of this stage is to study the transfer of heat from the fire to the structural member. As a result, the variation along time of the temperature distribution in the member is determined.

In order to accomplish with this analysis it is necessary to know the compartment temperature-time evolution regardless of whether it comes from the response described by a standard fire curve or from the data generated by a natural fire. Besides, the fire regime can have the form either of a continuous function or a discrete pairs of temperature-time points provided that they define with enough accuracy the progression of the heating source.

Two parts can be clearly differenced in the study of the heat transfer process of a structural member. The first part deals with the transfer of heat through the border from the heat source into the exposed surface of the member which is normally considered as boundary conditions with the combination of convection and radiation; the second part is about the heat transfer within the structural member governed by the heat conduction mechanism.

Governing equation

The heat transfer by conduction within the member is described by the Fourier differential equation:

$q = -\lambda \nabla \theta$ ……………………………………..(1.2)

where q is the heat flux vector per unit surface, λ is the thermal conductivity tensor

and θ is the temperature.

This phenomenon occurs due to the difference of temperature between two points of the member and the thermal energy is transferred through the solid or fluid between them but no matter moving is observed since it takes place at atomic levels. Integrating the law of conservation of energy into the Fourier equation results in:

$$\rho c \frac{\partial \theta}{\partial t} = -\nabla . q + Q \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1.3)$$

where ρ is the density, c the specific heat, t the time and Q the internal heat generation rate per unit of volume.

The specific heat of the material can be dependent of temperature, which introduces a non-linearity in this equation. Replacing equation (1.4) in equation (1.5) results in:

$$\rho c \frac{\partial \theta}{\partial t} = -\nabla . (\lambda \nabla \theta) + Q \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1.4)$$

which constitutes the heat transfer conduction equation to be solved subjected to an adequate initial state and boundary conditions.

Boundary conditions

First of all, the initial condition must be defined which implies specifying the temperature at which the element is submitted at the initial time.

Next, the boundary conditions of the element have to be established. The type of boundary condition applied to the fire exposed surface of the structural member is denominated Neumann boundary condition. It defines the normal derivative of the temperature as follows:

$$\lambda \frac{\partial \theta_m}{\partial n} = \dot{h}_{net} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1.5)$$

where n is the normal to the surface and $\dot{h}_{net}$ the net heat flux per unit surface.

The definition of the boundary conditions may adopt several forms depending on the heat source affecting the element. In the case of a structural member subjected to fire, EN 1991-1-2 (CEN 2002) includes in its Section 3 the thermal actions to be considered at the surface of a fire exposed member which are given by the net heat flux $\dot{h}_{net}$ produced by convection ($\dot{h}_{net,c}$) and radiation ($\dot{h}_{net,r}$).

$$\dot{h}_{net} = \dot{h}_{net,c} + \dot{h}_{net,r} \quad (\mathrm{W}/m^2) \ldots\ldots\ldots\ldots\ldots (1.6)$$

Except for the initial stages of the fire, radiation is the heat transfer mechanism governing the process. It does not require any medium for transferring the heat since it takes place by means of electromagnetic radiation waves which propagate through the vacuum.

The net radiative heat flux per unit area is defined by the next equation:

$$\dot{h}_{net,r} = \phi . \varepsilon_m . \varepsilon_f . \sigma . [(\theta_r + 273)^4 - (\theta_m + 273)^4] \quad (\mathrm{W}/m^2) \ldots\ldots\ldots\ldots (1.7)$$

where:

- ϕ is the configuration factor

- ε_m is the surface emissivity of the member

- ε_f is the emissivity of the fire

- σ is the Stephan-Boltzmann constant, equal to 5.67×10-8 W/m2K4

- θ_r is the effective radiation temperature of the fire environment [ºC]

- θ_m is the surface temperature of the member [ºC]

As observed, a configuration factor is included in the formulation which allows keeping the emissivity of the surface and the fire constant while varying radiative heat flux levels. According to EN 1991-1-2 (CEN 2002) Section 3, a conservative option is taking ϕ=1 but a more realistic value can be obtained by the method in its Annex G to account for the so called position and shadow effects.

The fire emissivity is generally taken as ε_f= 1.0. For the rest of emissivity values and according to EN 1991-1-2, Section 3, ε_m = 0.8 may be chosen unless a specific value is included in the material related fire parts of the same European code. In fact, EN 1994-1-2 in its Section 2 recommends a value of ε_m= 0.7 for the coefficient of steel and concrete related to the member surface.

Convection is the other heat transfer mechanism acting simultaneously with radiation. In this case, the thermal energy transfer takes place due to the motion of a fluid and, thus, mass transfer is observed.

The net convective heat flux component should be determined by:

$\dot{h}_{net,c} = \alpha_c.(\theta_g - \theta_m)$ (W/m^2) ………………..(1.8)

where:

-α_c is the coefficient of heat transfer by convection [W/m2K]

- θ_g is the gas temperature in the vicinity of the fire exposed member [ºC]

- θ_m is the surface temperature of the member [ºC]

For the coefficient of heat transfer by convection α_c, EN 1991-1-2 in its Section 3 recommends different values in function of the nominal fire curve chosen which can be consulted in Table 1.2 of this document. These coefficients are not an intrinsic characteristic of the surrounding fluid, but they are parameters obtained through a series of experiments taking into consideration several variables such as the geometry of the exposed surface of the structural element, the fluid properties or the origin of the fluid movement and its velocity.

Jointly with equation (1.6), equation (1.8) is also non-linear, in this case, as a consequence of the radiative boundary condition which implies the consideration of a non-linear term of the temperature. All the nonlinearities associated to the problem justify the habitual use of numerical methods to solve this type of heat transfer analysis.

Resolution of the heat transfer problem

With regard to the process of solving the heat transfer problem, it is worthy to mention that, generally, the calculation of the thermal response of a structural member can be decoupled from the computation of the structural response provided that the geometry of the structure does not have substantial changes during the period considered.

In order to solve the heat transfer problem and obtain the evolution of the temperature field with time, numerical modelling software is often used. Models developed with generic finite element analysis programs such as ABAQUS or with other programs more specialized in structures in fire like SAFIR can be employed.

Addressing the resolution of the differential heat transfer equation with an analytical approach can become a complicated and tedious work. Only for those cases where the heat transfer problem can be reduced to a sectional problem, analytical models are a reasonable option.

1.2.3. Structural analysis

In order to obtain the fire resistance rating of a member, the last step is to carry out the pertinent structural analysis. This step takes as a starting point the previously determined field of temperatures for the period of fire exposure considered. At every time step of the calculation process, the material temperature dependent constitutive equations are updated to account for the actual strains and stresses of the element. Finally, the FRR will be determined as the instant of time when the member collapse under a certain applied load.

In the field of CFST columns, several three-dimensional models, sectional models and also analytical models have been developed in the last decades with the aim of simulating the fire behavior of this type of composite columns.

In the next chapter, an extensive review of the state of the art which covers analytical, numerical and experimental investigations on CFST columns will be accomplished with the aim of acquiring the specific knowledge for dealing with the analysis of concrete filled tubular columns in fire.

2 STATE OF THE ART

In this chapter, the current state of the research in the field of the fire behavior of concrete filled tubular columns is reviewed, covering from the more simplified models to the complex advanced numerical models and the experimental investigations which have contributed significantly to the determination of the fire resistance of CFST columns.

2.1 Overview

Steel hollow structural sections are gaining popularity due to the number of advantages they offer over other shapes. When used in buildings, these columns have to satisfy fire resistance requirements prescribed in building codes. This is because fire represents one of the most severe environmental conditions to which structures may be subjected, and hence, the provision of appropriate fire safety measures for structural members is an important aspect in the design of high-rise buildings. The required fire resistance ratings, as per building codes, can be anywhere in the range of 1 to 4 hours and depends on a number of factors such as type of occupancy, height of the building and building location. HSS sections, on their own, have fire resistance of about 20 to 25 minutes and hence have to be provided with some kind of fire protection, to achieve required fire resistance ratings Lie and Kodur, (1996).

The importance and wide spread usage of CFST columns in structures have attracted many investigations from researchers across the globe for the past three decades. A number of scientific research has been done in North America, Europe, China, Japan and Australia. All these investigations witnessed that studying the thermal and mechanical behavior of CFST columns at elevated temperature is a complex phenomenon as it is affected by a number of different parameters.

Now a days, performance-based design methods are increasingly used for fire resistance assessment of structures. When modelling the response of a structure up on subjection to fire, it is paramount to accurately and efficiently determine the temperature development within structural members Gardner 2006 . This is owing to the fact that temperatures have a significant influence on material properties (e.g. strength, modulus of elasticity and deformation capacity) of structural elements. Therefore, the resolution of heat-and-mass transfer problem and its application to the sections of CFST members, is the first step to the fire resistance of the CFST columns.

The time-dependent cross-sectional temperature field can be obtained by using theoretical-analytical methods Lie 1984, Wang & Tan 2006, or advanced numerical models which make use of finite element analysis Ding & Wang 2008, Hong & Varma 2009. Also there are some FE soft wares developed in order to help with the calculation of parameter temperature field of the cross section, the general purpose FE softwares like ANSYS, ABAQUS, DYNA, ATENA, and FE softwares specifically developed for thermal and structural analysis (Sterner & Wickstrom 1990, Franssen 2003) can be mentioned. Moreover, the remarkable role on this field was the work done by Lie and co-workers Lie 1984, Lie & Chabot 1990, Lie & Irwin 1995, who derived equations for the temperature of the concrete core and steel shell based on Dusinberre 1961 on heat transfer calculations by finite differences.

The second approach on the study of CFST columns in which various researchers have concentrated is the constitutive modelling of materials- concrete and steel at ambient and elevated temperatures. Different authors have made their review on various models which are available in the literature. Lie & Purkiss 2005 made a significant contribution by studying the constitutive material properties of concrete from various models, and proposed a new model by comparing their discoveries with the models of Anderberg & Thelandersson 1976 , Schneider 1986,1988, and Youssef & Moftah 2007. After reviewing the constitutive properties of concrete and steel reinforcement, they developed a model which is able to predict the changes in the mechanical properties of concrete considering the effect of confinement at elevated temperatures. Youssef & Moftah also proposed two analytical models for confined concrete at high temperature based on the models of Scott et al 1982 and Mander et al 1988, at room temperatures. As most of the above investigations are done either on plain concrete or bar reinforced concrete, there is a scarcity of information on the constitutive law for SFRC at elevated temperatures.

The other focus of the researchers is on full scale experimental studies and development of analytical methods for estimating the fire resistance of CFST columns. Globally, more than 300 full scale standard fire tests have been carried out on CFST of various types. The main contributors to the available test database for concrete-filled SHS are the National Research Council of Canada (NRCC) and the Comité International pour le Dévelopment et l'Etude de la Construction Tubulaire (CIDECT). But, most of the experiments performed in the past few decades has used normal strength concrete, while the current trend is using high strength of concrete Rush et al 2010 .

Similarly, with the technological development in computers and soft wares in the recent decades, the numerical modelling of structural members for fire resistance using FE has shown significant interest among researchers from various parts of the world. Full scale experimental tests are costly taking much resources and time, with the help of numerical investigations the sensitivity of the structure to various parameters can be assimilated easily. To mention some who contributed in the numerical simulations of CFST columns are, Yu et al 2007, Hui et al 2009, and Wang et al 2013. Despite a number of difficulties in modelling, their results were in a close agreement with the experimental outcomes.

In the next sub sections of this chapter a detailed revision of the state of art on CFST columns subjected to fire is presented. The main experimental investigations done by researchers are presented and also the main models which are found in the literature are explained, covering both analytical models and numerical models. In addition, the available simple calculation models and design guidelines are included. Moreover, the mechanical properties of fiber reinforced concrete at elevated temperature including the pure shear properties are included.

2.2 Experimental investigations

The first time that the behavior of CFST columns under fire, experimentally investigated was between the years 1974 and 1982 by means of a research line sponsored by CIDECT (Comité

International pour le Développement et l'Etude de la Construction Tubulaire) with the conduction of several experimental campaigns Cometube 1976, Kordina & Klingsch 1983.

In the CIDECT research project 15A, Cometube 1976, 75 tests on protected and unprotected CFST columns of square cross-section (just one circular CFST columns was included) were carried out. Amongst them 6 columns were tested as hollow steel tubular columns, 19 with bar-reinforced concrete as infill and the remaining 50 columns were filled with ordinary concrete. All the columns were 3600 mm long but the external dimensions varied from 140 to 225 mm. The experiments were performed at Maizières-les-Metz and Champs-sur-Marne (France).

The effect of eccentricity was the new aspect to study introduced in the CIDECT research project 15 B, Grandjean et al 1980. In this campaign, circular and square CFST columns combining three types of infill (plain, bar-reinforced and steel-fiber reinforced concrete) were tested. A total of 86 tests were carried out in the framework of this program. In this case, the sectional dimensions of the square columns varied from 140 to 350 mm and up to 406.3 mm in the case of the external diameter of the circular specimens. The length of the columns was 3600 mm.

The report of the CIDECT research project 15 C, Kordina & Klingsch 1983 included the results of a series of 74 fire tests of different types of composite columns. In this case, the typologies tested were: CFST columns, hot rolled open section columns embedded fully in concrete and cold formed C- type section columns filled up with concrete and conventional solid steel columns. Five contrast fire tests were additionally conducted in different European laboratories. With regard to the CFST columns studied, they had principally square section and were filled with bar-reinforced concrete. A total of 26 specimens with lengths ranging from 3700 to 5700 mm were tested at elevated temperatures in the facilities from Brunswick University (Germany).

In 1991, an experimental campaign was performed in Borehamwood (UK) dealing with the fire resistance of circular CFST columns Wainman & Toner 1992. Three different diameters were considered, ranging from 244.5 mm to 355.6 mm. All the columns were filled with plain concrete and had an exposed length of 3100 mm.

In 2004, in the frame work of CIDECT research project 15R Renaud & Kruppa 2004, an experimental research was carried out in France in (Centre Technique Industriel de la Construction Métallique, CTICM), in order to study the fire resistance of an eccentrically loaded CFST columns. In total, four columns were tested: two of them of circular shape and the other two of square section. All columns had bar-reinforced normal strength concrete as infill and a total length of 3450 mm, although only 3100 mm were exposed to fire. The eccentricities used were 0.75 and 1.5 times the cross-sectional dimension. The experimental results from this research project was used as a basic reference for the development of a simplified calculation method for eccentrically loaded columns developed within the CIDECT project 15Q Renaud et al 2004.

Similarly, a successful experimental results were obtained regarding the CFST columns in fire, which were carried out by various researchers Lie & Chabot 1992, Kodur & Lie 1995, from the Institute for Research in Construction at the NRCC (National Research Council of Canada) between the years 1982 and 1994. They tested CFST columns of circular and square sections filled with plain, bar-reinforced and steel-fiber reinforced concrete. The cross-sectional dimension of the columns varied from 141.3 to 406.4 mm, the steel tube thicknesses ranged from 4.78 mm to 12.70 mm and the columns had a constant length of 3810 mm. All the columns were subjected to concentric axial load and generally tested under fixed end conditions. Approximately 2.3% of reinforcement was used for the bar-reinforced concrete and the percentage of steel fibers in the concrete mix for steel-fiber reinforced columns was 1.76% by mass.

In 2005, there were a series of experiments in NRCC again, addressing the fire behavior of CFST columns filled with high strength concrete Kodur & Latour 2005. Six circular CFST columns with diameters ranging from 219.1 mm to 406.4 mm and two square CFST columns of width 203.2 mm were tested in this experiment. The length of the column specimen was 3810 mm similar to the previous experiments. The concrete strength at the age of 28 days varied from 68.4 MPa to 90.5 MPa. Reinforcing bars or steel fibers were used as reinforcement in some of the specimens, in order to improve the fire performance of high strength concrete.

In the early 2000s, the research group headed by professor Han in China took honor in the field of fire tests conducted on CFST columns due to the successive campaigns carried out Han & Huo 2003. The main aims of the studies were concentrated on residual strength of CFST columns of CHS and SHS filled by plane concrete under monotonic or cyclic load after exposure to a standard fire. Outcomes of the experiments which were conducted in Tianjin (China) are presented in a number of papers. The main object of the studies were stub columns of 400 mm length (three times the diameter) and D/t ratio equal to 27.7, after 90 minutes of fire exposure.

The same research group in the University of Fuzhou Han et al 2003a also tested the fire resistance of slender CFST columns. In this program, 13 slender CFST columns of circular section were tested, 5 of them with external protection. The parameters investigated were the diameter of the cross-section varying from 150 to 478 mm, the steel tube wall thickness ranging from 4.6 to 8 mm, the fire protection coat thickness (0-25 mm) and the load eccentricity ratio (0-0.6). All the specimens of the column had a length of 3810 mm and two batches of concrete with compressive cube strength of 39.6 and 68.8 MPa were used.

The research group from the University of Seoul (Korea) Park et al. 2008 carried out tests on square CFST columns under both concentric and eccentric axial loads. Seven specimens of 300×9 and 350×9 mm section and 3500 mm long were tested in the first research Kim et al 2005. They were subjected to concentric load and different effective heating lengths. Tests in another 12 square CFST columns of 300×9mm section Park et al 2008 completed the study. These specimens were tested under concentric load and filled with normal strength concrete.

In 2009, a series of fire tests on high strength self-consolidating concrete filled steel tubular stub columns were carried out by Lu et al 2009. Six columns of square section with varying cross-section size, load case (concentric or eccentric load) and load level were tested. The cylinder compressive strength of concrete at 28 days was 90 MPa. All the specimens, which were 760 mm long, were tested in the Civil Engineering Laboratory at Monash University (Clayton, Australia).

In the last six years, several experimental programs led by Professor Romero from Universitat Politècnica de València (Spain) have been conducted on the fire resistance of CFST columns Romero et al 2011, Espinos et al 2015a, Romero et al 2015. These wide range of experiments has addressed the fire behavior of CFST columns of different cross-section shapes and configurations, various load levels and eccentricities and different types of infill and reinforcement. All the fire tests conducted by the group were tested in Valencia (Spain), in the facilities of AIDICO (Asociación de Investigación de las Industrias de la Construcción – Instituto Tecnológico de la Construcción). Also, several room temperature tests were conducted by the group at the Universitat Jaume I in Castellón (Spain).

In 2009, Romero et al. 2011 presented the results of an experimental program on slender axially loaded CFST columns subjected to fire. The aim of the testing program was to study the effects of three main parameters on the fire behavior of these composite columns: concrete strength (f_c), type of concrete infill (plain, bar-reinforced and steel fiber reinforced) and load level (μ). Normal (NSC) and high strength concrete (HSC) mixtures were employed, on the sixteen fire tests of the axially loaded columns. All the specimens had a circular cross-section of 159×6 mm and a length of 3180mm.

In line with the experimental investigations carried out by the research group headed by Professor Romero, an eccentricity of the loads was focused in a series of tests performed by Moliner et al. 2013. Its objective was to assess the fire response of eccentrically loaded slender circular CFST columns with normal and high strength concrete. For comparison purpose, the length and cross-sectional dimension of the specimens was kept constant with the previous experiment, Romero et al. 2011. A total of 24 fire tests were carried out in this case.

In 2014, the research group from Universitat Politècnica de València (Spain), come up with a novel idea of experimental research on concrete filled elliptical hollow section (EHS) columns. Until that time, no published works on elliptical CFST columns fire tests could be found in literature. The first experimental test on slender EHS columns was carried out by Espinos et al 2014. Totally, 12 specimens of column were tested, six of them at ambient temperature and the remaining six at elevated temperature. The test was focused on the influence of load eccentricity and type of infill (plain concrete or bar-reinforced concrete). Cold formed elliptical steel hollow sections with external dimensions 220x110 mm and a wall thickness of 12 mm were studied. The length of the columns tested at room temperature was 135 mm, while the length of the fire tested specimens was 3180 mm.

Taking the research line further, Espinos et al. 2015a extended the experimental program on CFST columns of elliptical and rectangular shapes under fire conditions. The section size of the analyzed 6 elliptical columns was 320x160 mm with a wall thickness of 12.5 mm. 12 rectangular columns with two groups of different cross-sectional dimensions of 2510x150 mm and 350x150 mm were tested with the same wall thickness of 10 mm for all. All the columns were 3180 mm long. The parameters which considered in the influence of fire response of these columns were the load eccentricity, percentage of reinforcement (considering both major and minor axis) and the shape of the cross section.

Subsequently, another series of tests was conducted on circular and square slender CFST columns under large eccentricities by Espinos et al 2015b. A total of 12 specimens were tested: 6 with circular cross-section and other 6 with square cross-section shape. For each shape two cross-section sizes were taken for the test. For circular columns, 193.7x8 mm and 273x10 mm sections were tested, while 150x8 mm and 220x10 mm sections were used for the square columns. The applied eccentricities were 0, 0.5, and 0.75 times the cross-sectional dimension. All the specimens were bar-reinforced and had a length of 3180 mm. The main aim of the study was to determine the influence of the cross-section shape, load eccentricity and percentage of reinforcement on the fire response of CFST slender columns with large eccentricities.

More recently, Romero et al 2015 got involved in the study of an innovative cross-sectional configuration of CFST columns. He performed an experimental investigation on the behavior performance of six double-tube concrete filled steel tubular columns with different combination of outer and inner steel tube thicknesses, both at ambient and elevated temperatures. The room temperature behavior was assessed first, and then the experiments under fire conditions were followed in order to compare the behaviors between these two situations. The effect of filling the inner tube with concrete on fire resistance was evaluated. In addition, the influence of different combinations of concrete strengths was investigated. The length of the columns tested at ambient temperature was 3315 mm while the fire tested specimens were 3180 mm long.

In continuation to the line of research, an experiment was done on the applicability of new materials to CFST columns in fire. A series of tests were carried out using stainless steel and self-consolidating concrete by Han et al 2013. Five full-scale tests on square and circular CFST columns subjected to axial compression were tested. The steel tube was made of austenitic stainless steel and the infill was self-consolidating concrete. The main parameters of study were the load level and sectional type and dimensions. The total height of the CFST columns was 3600 mm, and only 3000 mm was exposed to fire. The tests took place in Tianjin Fire Research Institute (China).

The main focus of study has been done only on the performance of CFST columns subjected only to uniform fires in the last few decades. As a results there is a scarcity of knowledge for the behavior of columns exposed to three sides in fire, which is the common scenario in practice. Therefore, Yang et al 2013b conducted an experimental study to investigate the real behavior in the case of non-uniform exposure on square CFST columns. In order to observe the effects of the number of exposed sides, load ratio and load eccentricity, six columns were tested to failure. Four of the columns had 3 sides exposed to fire and the remaining only one side. The experiment was continued with rectangular CFST columns Yang et al 2013a, with three full scale tests. Two columns were exposed to fire in three sides, while the remaining one just only one face. The column specimens had the same length as the previous test with dimensions of 200x300 mm. All the nine experiments took place in Suzhou University of Science and technology (China).

Regarding the post-fire behavior of CFST columns some experimental results can be found in literature. Tao et al 2011 carried out a series of push-out tests on 64 columns previously exposed to the ISO 834 fire curve for a determined period of time (90 or 180 minutes). The main objective of the study was to reach at some conclusions about the post-fire bond between the steel tube and the concrete core in CFST columns. Another 12 tests were carried out on unheated specimens for a reference purpose. Concrete filled tubular columns of both shapes, square and circular were studied and circular specimens showed much higher bond than square ones. The tests results indicated the relevant influence of the fire exposure.

Similarly, Rush et al 2015 showed the results of his experimental investigation on post-fire residual compression tests on protected and unprotected CFST columns. Overall, 25 columns were tested from which 19 were tested after being heated and the rest 6 were tested before heating and used as control specimens. CFST columns of both square (7) and circular (18) cross-section shape were used. The main objective of the experiment was to assess the effect on the post-fire residual capacity of the cross-section shape, the steel tube wall thickness, the heating curve and the fire protection applied.

2.3 Numerical studies

The experimental investigations for being very much expensive and time and labor consuming, it has led to the development of numerical studies which is relatively cheap and takes less time. Also, through the latest technological advancement in computers, numerical modelling is catching the interest of various researchers. A large number of finite element computer codes are available nowadays, which can be used to solve the nonlinear heat transfer problems. Some of the most commonly used are FIRES-T3 Iding et al 1977, developed at the University of Berkeley (California, USA), TASEF Starner & Wickström 1990, from Lund Institute of Technology (Sweden) and SAFIR Franssen 2003 developed in University of Liège (Belgium). In addition, to these specifically developed for the fire analysis of structural members, general purpose finite element programs exist such as ABACUS, ANSYS, DIANA, ATENA, among others, which can be also used for solving the thermo-mechanical problem under study.

In what refers to CFST columns, several numerical studies have been carried out worldwide. Two categories of models are defined in the study of numerical models of CFST columns in fire, global structural models and isolated member models. For concrete filled tubular columns, most of the published works focus on the study of CFST columns in fire as isolated columns with explicitly well-defined boundary conditions which do not change during the fire exposure time. Nevertheless, some relevant global structural models dealing with the study of CFST columns in fire within a frame can be found in the literature. The main numerical investigations which can be found in literature are hereafter reviewed.

Wang 1999 presented a global model to evaluate the effects of structural continuity on the fire resistance of CFST columns. A FEA computer program was developed by the author to simulate the structural response of composite frames at high temperatures. A finite element analysis was executed for obtaining cross sectional temperatures and the mechanical response in fire. For the thermal analysis, two-node fiber elements were used, with each element representing a circular slice of the cross-section. Thermal properties of the materials and stress-strain relationships from EN 1994-1-2 (CEN 2005c) were used and the thermal resistance at the steel-concrete interface was neglected, since the cross-section was treated as a continuous medium.

In this research line, Bailey (2000) also presented a global structural model for the study of the effective length of square CFST columns in fire. A finite element model for simulating square CFST columns at room and high temperatures was programmed to be incorporated to a pre-existent computer model. One-dimensional two-node finite elements with 7 degrees of freedom at each node were employed and the cross-section was subdivided into a number of square or rectangular segments. The temperature dependent material constitutive models from EN 1994-1-2 (CEN 2005c) were adopted and the concrete tensile strength was updated at each step and calculated as the 10% of the reduced compressive strength. As a first step, the two dimensional thermal computer package TFIRE was used to obtain the temperature distribution through the cross-section. Local buckling could not be directly reproduced since fiber elements were employed. Therefore, a specific local buckling element was employed in the model. The author recommended the next effective length design values for concrete filled tubular columns at fire limit state: for columns continuous at both ends 0.75L; for columns continuous at one end 0.8L (continuous at their base); and for columns with pin foundation 1.0L.

With regard to numerical models for the fire simulation of isolated CFST columns, fiber models (one-dimensional), sectional models and three-dimensional models can be found in literature. The trend followed by most authors is developing sectional and three-dimensional models capable of reproducing the remarkably nonlinear behavior of these members at high temperatures. Nevertheless, fiber models are the most efficient in terms of computational cost although sometimes some assumptions are made for the sake of simplicity. Since fiber models (one-dimensional) are the most efficient from the computational point of view, it was the option chosen for the two structural models presented above where part of a structure is modeled. In the following paragraphs a details for each type of models are presented.

Fiber models (One-dimensional)

An advanced numerical model was developed by Renaud et al 2003 using beam column elements with an updated lagrangian formulation to simulate the fire behavior of steel-concrete columns taking into account the interaction between the hollow steel section and the concrete core. The thermal and the mechanical behavior of the columns were assumed to be uncoupled. The authors modeled a CFST column combining in a parallel way the next elements: one beam-column element for the steel hollow section with two nodes and three degrees of freedom at each node; a second element of the same characteristics representing the concrete core; and a third element acting as a longitudinal shear link between them along the whole length. Each element had its corresponding fiber discretized section. A uniform temperature was assumed over the entire column length and the cross-section thermal analysis was carried out by a finite differences method. Thermal and mechanical material properties from EN 1994-1-2 (CEN 2005c) were used. Although slip between the two components was taken into consideration, the gap phenomenon between steel and concrete was neglected. The authors found that when the whole response was analyzed from the point of view of the axial displacement along time, the column elongation was overestimated at the end of the test, even when slip between steel and concrete was considered. However, the model proved to give a good estimation of the fire resistance time.

A simplified fiber model for square CFST columns under concentric loads was developed by Chung et al 2008 and later extended to eccentric loads Chung et al. 2009. Square elements were used to discretize the section. The thermal analysis, carried out by means of finite differences, was decoupled from the mechanical analysis. The thermal and mechanical properties of the material were taken from Euro code 4 (CEN 2005c). The authors make some assumptions like ignoring the local buckling of the steel tube, not considering the steel-concrete interaction and neglecting the initial imperfection of the column.

Yang et al 2008a developed NFEACFST, a finite element model for CFST columns which considers the column length sub-divided into two-node finite elements. Circular and square columns were modeled with the composite cross section discretized in small blocks. This model covers properties of all the stages including the room temperature, heating, cooling and post-fire loading. It also considered some simplifications such as neglecting the steel tube local buckling or ignoring the relative slip existing between the steel tube and the concrete core at elevated temperature.

Similar to the above research, Jeffers & Sotelino 2009 presented a three-node fiber heat transfer element accounting for transverse and longitudinal temperature variation in a structural member for modeling the three-dimensional response of structures in fire. However, the authors did not focus on developing a model for the fire behavior of CFST columns but rather they developed a heat transfer element which could be used in a multitude of elements and more complex structures. The effort was made on solving the heat transfer problem and making the element compatible with any beam-column element so that once the finite element was implemented in commercial software such as ABAQUS, a sequentially coupled thermal-mechanical analysis could be run using elements available in the software.

Sectional Models

Schaumann et al 2009 developed a sectional numerical model to predict the fire behavior of high strength CFST columns with different type of reinforcement. The program BoFIRE, developed by the authors, was used to implement the model and four-node iso parametric elements were used. It was able to reproduce the thermal and structural behavior of steel and composite structures at elevated temperatures with the appropriate temperature dependent material properties and considering the actual temperature distribution. The analysis was carried out by means of an incremental procedure where thermal and structural responses were coupled at every step. Although the finite differences method used for the cross sectional thermal analysis reproduced the temperature field with great accuracy, the whole model was not able to represent local effects such as the contact mechanism that occurs under fire and, consequently, the mechanical response obtained was a little away from the actual one.

Three-dimensional Models

In 2003, a three dimensional finite element model was developed by Zha 2003 for CFST columns. First, the computer program FIRES-T3 Iding et al 1977 was used to compute the cross sectional temperature. Once the temperature distribution was known, a time-dependent thermal-stress analysis was performed using DYNA3D, a three dimensional nonlinear finite element code for solid and structural mechanics developed by the author. Eight-noded solid elements were used in model. Material models for elevated temperatures were employed: the model proposed by Schneider 1988 for the concrete core and the equations presented by Witteveen et al 1977 for the steel tube. Since circular columns where considered, modelling a quarter of the section was enough due to symmetry. A specific value of 2L/1000 was adopted for the initial imperfection of the composite column. In this work, no comparison with experimental data for calibration was included and only the fire resistance times obtained were compared to those given by the pertinent codes.

Using the commercial finite element software ANSYS, Ding & Wang 2008 developed an advanced three-dimensional model for circular and square CFST columns in fire. The authors included some significant aspects which has been neglected often by various researchers for simplicity. These aspects were the concrete moisture content, the gap thermal resistance at the steel-concrete interface and the relative slip between the two components represented in this case by a surface to surface contact. For the heat transfer analysis 2-D solid thermal elements were considered, whereas for the structural analysis eight-noded 3-D solid elements were used. Material properties at elevated temperatures were defined according to EN 1994-1-2 (CEN 2005c). The model was able to produce satisfactory predictions from the point of view of the fire resistance rating.

Hong & Varma 2009 developed another three-dimensional model using the FE software ABAQUS, for predicting the standard fire behavior of square CFST columns. However, these authors modeled the steel-concrete interface as perfect contact, assuming that no heat loss exists at this boundary. In this model, the fire dynamics analysis was considered as a prior step to the sequentially coupled thermal-stress analysis which transformed the model in a three-step procedure. The fire dynamics analysis was carried out by means of a computer program FDS (Fire Dynamics Simulator), developed by researchers from the NIST Building and Fire Research Laboratory. From this analysis, a temperature time curve was obtained to be the input of the thermal analysis model. For the heat transfer and stress analysis, eight-node solid elements were used to model the concrete core and four-node shell elements for the steel tube. For the specimens with reinforcing bars, two-node truss elements were used for the longitudinal elements. After evaluating the response of several materials mechanical models, Lie & Irwin 1995 for concrete and Poh 2001 for steel were adopted.

Lu et al 2009 studied the fire response of high strength self-consolidating CFST stub columns of square sections using the commercial program ABAQUS, complementary to an experimental program carried out by the same authors. Again, a sequentially coupled thermal-stress analysis procedure was proposed. Linear four-node shell elements were employed for the steel hollow section whereas for the concrete core linear three dimensional eight-node solid elements were used. The thermal properties of high strength concrete given by Kodur 2007 were used except for the thermal expansion coefficient for which the model from Lie 1994 was adopted for both concrete and steel. The mechanical models implemented were the uniaxial stress strain model at elevated temperatures from Lie 1994 for the steel tube; and the compressive uniaxial stress-strain relation proposed by Han et al 2003b with a concrete damaged plasticity model for the core. For the thermal resistance at the steel-concrete interface, a heat contact conductance parameter of 100W/m^{2}k was used. This model was intended to analyze the failure mechanism of these composite columns in fire and to investigate aspects like the concrete fracture energy, the steel-concrete contact, the load distribution between components or the local buckling of the steel tube.

In 2010, a research group led by professor Han at Tsinghua University (China), developed a realistic finite element three-dimensional model in ABAQUS to simulate a set of experiments of concrete filled steel tubular stub columns under various thermal and mechanical loading conditions Song et al 2010. This research was done on similar way to the previous research which was carried out by Yang et al 2008a. The model was used to study all the stages during fire (room temperature, heating, cooling and post-fire), with the help of stress-strain relationships for each stage. The model was meshed by means of four-noded shell elements for the steel tube and eight-noded brick elements for the concrete core. In order to model the interaction between the two surfaces at the steel-concrete interface, a contact pressure model in the normal direction and a Coulomb friction model in the tangential direction were adopted.

An advanced three-dimensional model was developed by Espinos et al 2010 taking into account some of the aspects ignored by other researchers. It focused on the fire behavior of unprotected CFST columns of circular shape. A sequentially coupled thermal-stress analysis was modeled using the program ABAQUS. Once the temperature field was obtained the nodal temperatures were the input of the mechanical analysis in form of predefined field. Both the steel and concrete were meshed with three-dimensional eight-noded solid elements, whereas the reinforcement was meshed with two-noded truss elements. The concrete mechanical model developed by Lie 1994 and the steel mechanical model from EN 1993-1-2 (CEN 2005b) were adopted. Sensitivity analysis was carried out for the model and it was validated against an experimental results. The slippage between the two components, the thermal conductance at steel-concrete interface, the moisture content of the concrete and the initial imperfection were some of the realistic consideration included in the model. As a result the model was able to produce accurate fire resistance results and the whole response of the column along time was well captured.

In the line of emergence of new shapes and configuration and special materials, also the same authors proposed a model for the performance behavior of elliptical CFST columns both at ambient and elevated temperatures Espinos et al 2011. The model was built on the previous work of circular CFST columns carried out by the same authors. The room temperature model for elliptical CFST stub columns was validated against test data and subsequently extended to evaluate their fire response. Due to the unavailability of experimental data at high temperature, the same material properties and parameters of the consolidated model for circular CFST columns Espinos et al. 2010 were adopted. The model was meshed with eight-noded elements using ABAQUS software and sequentially coupled thermal-stress analysis was employed. In addition a comparative and parametric studies were conducted to assess the effectiveness of the section shape compared to circular CFST columns and the influence of main parameters variations. According to the study, it showed that under certain concentric loads and only for stub columns, circular members showed a better fire comportment.

In the same line of new configurations, Lu et al 2011 developed a model for double-skin CFST columns with circular and square hollow section for both inner and outer tubes. ABAQUS package was employed to develop a sequentially coupled thermal-stress analysis. Interaction was considered to be a very important factor in the fire behavior of these columns. For the concrete the model proposed by Han et al 2003 was used and for steel Lie’s model 1994 was adopted. Once the model was validated, a parametric analysis was executed to study aspects like the capacity of inner and outer steel tubes, the fire protection or the sectional dimensions. Some design recommendations were given.

Wang &Young 2013 investigated the effects of using high strength steel in CFST columns. A three-dimensional numerical model was developed using ABAQUS and calibrated against tests results of CFST columns with normal strength concrete given the lack of experimental data regarding this material. A two-step analysis was modeled: first, a thermal model was executed and next a nonlinear structural analysis was completed. A parametric study was carried out to evaluate the influence of high strength steel with yield strength of 690 MPa on CFST columns. The fire resistance of the members with high strength steel showed a significant improvement with respect to the CFST columns whose steel tubes had yield strength of 275 MPa, but it did not occur when specimens under the same load level were compared.

Tondini et al 2013 also investigated the characteristics of high strength steel at elevated temperatures. A three-dimensional finite element model was developed in the SAFIR platform to represent numerically the fire behavior of some circular hollow sections tubes and concrete filled circular tubes that had been previously tested by the authors. The main purpose was to check the applicability of the reduction factors recommended by EN 1993-1-2 (CEN 2005b) and there were evidences of an overestimation of the fire resistance. However, given the limited number of experiments used in validation these conclusions were not as solid as desirable.

Recently, more study has been done on the study of post-fire behavior of CFST columns. As a result, Yao & Hu 2015 studied the cooling behavior and the residual strength of CFST columns subjected to fire. The authors presented a three-dimensional model developed in ABAQUS which covered all the stages of the complete fire. The analysis was sequentially coupled; first a thermal analysis was carried out and then the temperature file was introduced as a predefined field for the structural analysis. Heating conditions were varied, employing both nominal and natural fire curves. The model was validated against numerous experiments and a parametric study was developed. It was observed that the residual strength of CFST columns after natural fire exposure is generally affected by the fire duration time, cross-sectional dimension and slenderness ratio.

Similarly, a three-dimensional model was developed by Yang et al. 2015 in ABAQUS to simulate the post-fire behavior of slender bar-reinforced CFST columns. A sequentially coupled thermal – stress model was implemented. The model was validated against experimental data from tests carried out by the authors within the same study. The heating phase was according to the nominal standard fire curve and the cooling phase were included in the analysis. After calibration of the model, a parametric study was executed to identify the influence of key parameters on the post-fire behavior. The residual capacities were observed to be sensitive to heating time, cross-sectional dimension, slenderness ratio, material strength, steel tube to concrete area ratio and reinforcement ratio, whereas the buckling reduction factor was found to depend mainly on the slenderness ratio, the heating time and the cross-sectional dimension. Finally, a simplified design method was proposed for predicting the load-bearing capacity after exposure to standard fires.

2.4 Analytical approaches

The first published theoretical research about fire behavior of CFST columns is the work of Lie 1984. He derived a mathematical solution for computing the cross-sectional temperatures and also proposed a structural model which helps to calculate the mechanical response of CFST columns. Thus, given a certain curvature at column mid height, the axial strain is used as the iteration variable to satisfy equilibrium between the applied and resisting moment. With this procedure, the force-mid-height deflection relation is determined step by step during fire exposure in order to arrive at the maximum column load carrying capacity. The model developed by Lie was validated against the experimental results of circular Lie 1984, Lie 1994 and square Lie & Irwin 1995 CFST columns under fire. Specific formulations for the constitutive laws of steel and concrete at elevated temperatures were used. A discrepancy was observed between the measured data and the analytical axial deformation because of the thermal expansion and creep which can't be completely accounted for in this type of model.

Han 2001 presented a structural model to predict the fire resistance of CFST by means of an analytical procedure. Thermal dependent constitutive equations for steel and concrete were employed, taking into account the confinement effect in the concrete stress-strain relationships. Hence the model accounted for the physical and geometrical non-linearities existing in the fire behavior of CFST columns. The model was validated against tests of square and circular CFST columns. This theoretical model was used to provide information about the necessary fire protection measures for the CFST columns in the SEG Plaza in Shenzen, the highest high-rise building finished in China which includes CFST columns in its design.

Ghojel 2004, developed expressions for estimating the steel-concrete interfacial contact conductance of loaded and unloaded circular steel tubes filled with plain concrete, as a function of steel tube temperature. Before Ghojel most of the researchers had ignored the resistance to heat flow in the interface by assuming simplified boundary conditions.

Tan & Tang 2004 extended the Rankine method to predict the fire behavior of plain and reinforced CFST columns. The fire resistance of a CFST column is expressed in the form of the plastic squash load and elastic capacity at normal room temperature. Based on the same approach, Hu et al 2015 recently proposed a modified Rankine method considering the shear bond effect between the two failure modes. The conventional Rankine approach uniquely considered a linear interaction between the two failure modes and ignored their coupling interaction which results in a lower limit prediction in comparison with measured failure loads. Model results were validated against test data from different programs and also compared to the EN 1994-1-2 predictions.

A theoretical approach was presented by Wang & Tan 2006 for the heat transfer analysis of concrete filled steel circular hollow sections subjected to fire based on an analytical Green's function solution. The temperature field inside the composite domain and the heat flux at the fire and steel-concrete interfaces can be estimated using this approach.

Yin et al 2006 developed an analytical nonlinear model for circular and square CFST columns,. The first step of the model deals with the calculation of the temperatures by solving the corresponding heat transfer equations. Next, a mechanical analysis to obtain strain and stresses at elevated temperatures is presented, where the compression failure mode was exclusively considered, assuming no bending and straight columns at failure. In this case, for the concrete core the constitutive expressions by Li & Purkiss 2005 for normal strength concrete were applied; for the steel, strain-stress equations proposed by Lie 1994 were employed. For comparison purposes, the square and circular columns studied were selected to have the same cross-section areas for both steel tube and concrete core, concluding that columns with circular shape presented a slightly higher fire resistance.

2.5 Simple calculation models

In order to facilitate the use of CFST columns in structural design, it is necessary to provide simple calculation models which facilitates the determination of the member resistance both the room and fire resistance. The employment of this type of composite columns in actual constructions has increased in the last decades due to their inherent fire behavior and it has revealed the necessity of relying on a consistent simple calculation model for fire design.

A number of design guides on the calculation of the fire resistance of CFST columns have been developed. Among the various design guide lines developed, it is the CIDECT guide Twilt et al. 1996. In this guide line designers can find a number of design charts valid for the more commonly used cross-sectional dimensions, where the load-bearing capacity of the column for a certain fire exposure time is given as a function of its buckling length, cross-sectional dimensions and percentage of reinforcement.

In 2002, the Corpus Tubes guide Hicks & Newman 2002 was published following the CIDECT guide, in order to update the knowledge about the fire response of these composite columns in fire and provide design recommendations for structural designers.

Most of the simple calculation proposal methods are obtained by the main research group through extended and deep experimental and numerical investigations on the behavior of CFST columns under fire. Thus, some of these design methods have been included in the respective national codes to facilitate the use of CFST columns by practitioners.

As an example of this process it can be observed on the proposal of Han et al 2003a, which is incorporated in the Chinese Code DBJ13-51 (2003). It consists of a set of expressions to compute both the strength index of CFST columns without external protection in fire and also the thickness of external fire protection to apply in order to achieve a given FRR.

In Japan and Korea, designers opt for different approaches. In Japan the reference has been the simple formula given in the manual for the design of CFST columns presented by the Association of New Urban Housing Technology, while in Korea took relevance an empirical expression proposed by Park et al 2007, 2008 for CFST columns of square shape which was obtained by regression analysis based on previously established relationship between the fire resistance and the parameters width of the column, concrete strength and applied axial load ratio.

Rush et al 2012 has reviewed the current methods that exist for calculating the fire resistance of CFST columns and stated that the most common fire resistance approaches for the design of CFST columns are the ones currently recommended in North America and Europe.

In North America, the design equation proposed by Kodur and collaborators was integrated in its respective code, the National Building Code of Canada (NBCC 2005) and similarly incorporated by the American Society of Civil Engineers (ASCE 1999), AISC Steel Design Guide 19 (Ruddy et al. 2003) and ACI 216 (ACI 2007). These calculation method was proposed after carrying an extensive experimental investigations at NRCC. The applicability of the formula includes square and circular CFST columns with any kind of infill (plain, bar reinforced or fiber reinforced concrete) of both normal and high strength.

On the other hand, Euro code 4 Part 1.2 (CEN 2005c) is the reference code in Europe for the fire design of composite structures including CFST columns. Due to proximity and importance, the subject of this code is described in detail in the next sections. Three levels of design methods are followed: tabulated data, simple calculation models and advanced calculation models.

Tabulated data is the most simplest and immediate option for fire design contemplated by EN 4 Part 1.2 (CEN 2005c) for classical cross-section shapes. This option gives the minimum cross-sectional dimensions as well as the minimum reinforcement ratio and axial distance of reinforcing bars using tables in order to achieve certain FRR under a specific applied load level. It is the most conservative method and according to Rush et al. 2012 results predicted using this method are extremely safe.

The simple calculation model which is included in Euro code 4 Part 1.2 provides a general method of calculation of design value in the fire situation of the resistance under axial compression of composite columns, including CFST columns. A complete procedure can be found in its clause 4.3.5.1. First, the cross-sectional temperature field needs to be obtained and after discretizing the cross-section in elements, the design fire resistance is computed considering the contribution of all the components in the cross-section of the column. Different methods can be used to determine the temperatures in the cross-section such as one-dimensional heat transfer analysis or finite element analysis. Though, the calculation of the axial buckling load is straightforward, the determination of the cross-sectional temperature field can result a tedious procedure to be implemented in the daily practice.

Moreover, Wang 2000 developed a simple method for designers which avoids the necessity of calculating the cross-sectional temperature distributions explicitly. According to this method the load bearing capacity of circular CFST columns in fire was evaluated based on the squash load and the rigidity. The author carried out a parametric study based on the general approach of Euro code 4 Part 1.2 (CEN 2005c) to check the accuracy of their proposal. The information needed for its implementation is given through tables and can be used by linear interpolation.

Furthermore, the Annex H of the Euro code 4 Part 1.2 (CEN 2005c) presents a simple calculation model specifically developed for unprotected concrete filled tubular columns. It is based on a prior method proposed for CFST columns at ambient temperature Guiaux & Janss 1970. Renaud et al 2004, in the framework of CIDECT project 15Q developed a numerical investigation and pointed out the theoretical shortcomings of the method. Also, it was proved that this method produces unsafe results for columns with common levels of slenderness and its basis were doubted Wang &

Orton 2008. These measures led to the inclusion of correction in Annex H to point out that the applicability of the method was restricted to CFST columns with a value of relative slenderness equal or less than 0.5.

Practically, the specific methods for CFST columns was tiresome to apply despite its assumed simplicity and also produces unsafe results for slender columns. Therefore, more number of researchers started to follow another trend by studying the general approach of Euro code 4 Part 1.2 which is described in Clause 4.3.5.1. Some investigations were done in order to take this step further and to check the applicability of the general method particularly to CFST columns. As a result, the National Annex to EN 1994-1-2 was developed in France (AFNOR 2007) considering the effort and conclusions done by the CTICM group regarding the fire design of CFST columns. It was a clear example of the promotion of the use of CFST in structural designs through a research.

Parallel to this line, based on the general approach from EN 1994-1-2 Clause 4.3.5.1 (CEN 2005c), Espinos et al 2012 developed a simple calculation model for evaluating the fire resistance of unreinforced axially loaded CFST columns. In this method, the traditional calculation of the non-uniform cross-sectional temperature field is substituted by a given set of equations to compute the steel tube temperature and the equivalent concrete core temperature for a given FRR and as a function of the geometrical characteristics of the column. Concerning the mechanical model, the authors proposed appropriated flexural stiffness reduction coefficients for the calculation of the effective flexural stiffness in fire situation as a result of an extensive parametric study and its subsequent statistical analysis. This simple model extends the limitations of applicability of the general approach from Clause 4.3.5.1 to more slender columns. This method was validated against experimental results and proved to be generally on the safe side with prediction of reasonable accuracy. This simple model extends the limitations of applicability of the general approach from clause 4.3.5.1 to more slender columns. Later this simple calculation model was extended to bar reinforced columns of circular and elliptical shape cross-sections taking into account the reinforcement ratio Espinos et al 2013. Adjusted expressions were obtained for the innovative elliptical cross-sections even though the circular expressions were also proved to be valid.

In 2014, Yu et al 2014 also published a fire design proposal using the average temperature approach. Two average temperatures, one for the steel and another for concrete were considered. These average temperatures were calculated based on the equations developed by the authors after making a regression analysis and considering the cross-sectional dimensions. Then, a design method based on the room temperature method from Euro code 4 Part 1-1 (CEN 2004b) was presented. The proposal covered circular and polygonal sections and, according to the authors, showed acceptable accuracy in its predictions.

Recently, Imani et al 2015 provided a simplified analytical procedure for the calculation of the fire resistance of concrete filled double tube steel tubular columns subjected to any heating curve. The heat transfer analysis is calculated analytically and the axial load capacity is obtained using the temperature dependent properties from EN 1994-1-2. Simple assumption was made by the authors considering the outer steel tube temperature equal to that of the heating curve. The axial load capacity calculation procedure proposed was similar to the simple model for CFST columns included in Annex H of EN 1994-1-2, although a simplified step by step procedure was presented.

According to the authors more research is necessary to properly check the accuracy of the proposal as it verified against only one experiment.

2.6 Properties of fiber reinforced concrete column at elevated temperature

In recent years, the construction industry has shown significant interest in the use of fiber-reinforced concrete due to the advantages it offers over traditional plain concrete. The use of steel-fibers as reinforcement in plain concrete not only enhances the tensile strength of the composite system but also reduces cracking under serviceability conditions. Further, steel-fibers improve resistance to material deterioration as a result of fatigue, impact, shrinkage and thermal stresses. The improvements in material properties, which improve structural performance, have extended the use of fiber-reinforced concrete to applications in the area of fire.

A number of experimental investigations have been conducted up to date with the aim to observe the fire response of concrete composites. Particularly, the studies have focused on the effect of a type, shape and content of fibers on the mechanical properties of concrete composites, mostly compressive and tensile strength including elastic modulus. Namely, it concerns steel fibers, synthetic fibers and a mix of steel and polypropylene fibers which are widely used in the concrete industry. There are also a few investigations which deals with carbon fibers and glass fibers.

The mechanical properties that determine the fire resistance of structural members are the strength and deformation properties at elevated temperatures of the materials of which the members are composed. For concrete, the important mechanical properties are the compressive and tensile strength of the concrete and the deformations caused by load, creep and thermal expansion. These properties are usually expressed in stress-strain relations, which are used as input data in mathematical models for the calculation of the fire resistance of concrete members.

These mechanical properties of FRC at elevated temperatures were studied by a number of researchers. Kodur and Lie on their paper Lie & Kodur 1995 made a detailed experimental study on this topic which contributed a lot to identify the mechanical properties of FRC which can be used as an input data in the numerical models. Based on their experiment, the authors concluded that the compressive strength at elevated temperatures of fiber-reinforced concrete is higher than that of plain concrete. The compressive strength of fiber-reinforced concrete increases considerably till a temperature of approximately 400°C; whereas the compressive strength of plain concrete continuously declines with increasing temperature. Similarly they identified that the presence of steel fibers increases the ultimate strain and improves the ductility of a fiber-reinforced concrete member. In addition they found that fiber-reinforced concrete develops higher restraint stresses than plain concrete. Overall, fiber-reinforced concrete exhibits, at elevated temperatures, mechanical properties that are more beneficial to fire resistance than those of plain concrete.

The increasing utilization of FRC in the concrete industry is motivated by the physical and mechanical properties of the material which contribute to traditional concrete elements and structures various economic benefits. The extensive knowledge of mechanical properties of FRC exposed to elevated temperature seems to be decisive for a wider utilization of the material. Most of experimental investigations are conducted on test specimens at ambient temperature after high temperature exposure and only a few is performed on heated test specimens. Novák & Kohoutková (2017) presented a paper providing information on the behavior of different types of Fibers in fiber reinforced concrete at elevated temperatures. According to the authors the combination of steel and synthetic fibers represents a promising alternative how to ensure good toughness of a concrete composite before heating and improve its residual mechanical behavior and spalling resistance as well as the ductility after heating.

2.7 Pure shear properties of fiber reinforced concrete column

Concrete is generally a brittle material and reinforcement is added in order to improve some of the mechanical properties of it. As part of these fiber reinforcement with sufficiently high tensile strength and modulus of elasticity is added to concrete to enhance the ductility and the fracture energy, thus concrete gain ability to carry load after the propagation of crack.

Real applications of fiber reinforced concrete are frequently arranged to carry shear forces. However, there was no realized experiment investigating the performance of fiber reinforced concrete in pure shear mode without any flexural effects. In order to fill this gap, Václav Ráček et al. 2015 investigated a new experimental arrangement in which a large-scale tube fiber reinforced concrete is subjected to pure torsion. Pure shear was produced on the thin walls of the simply supported fiber-concrete by the application of torque. The objective of the experiment was to measure not only the ultimate strength but also the descending post-peak of the torque-twist diagram. This descending part of the diagram is very informative for composites with fibers. Experimental results bring new and important information essential for comprehensive understanding of fiber-concrete. These results enable to derive the stress–strain relation in the whole range of stress-strain diagram in the pure shear mode.

2.7.1.1 *Torsional strength values and behavioral curves*

The loading process is controlled by twist (the angle of twist $\emptyset$) produced by the loading couple and corresponding torque (the torsional moment M_t) is recorded. The angle of twist per unit length $\theta = \frac{\partial \emptyset}{\partial x}$ is calculated and values of the torque M_t as a function of θ can be plotted. Typical shape of the M_t - θ diagram is shown in Figure 2-1.

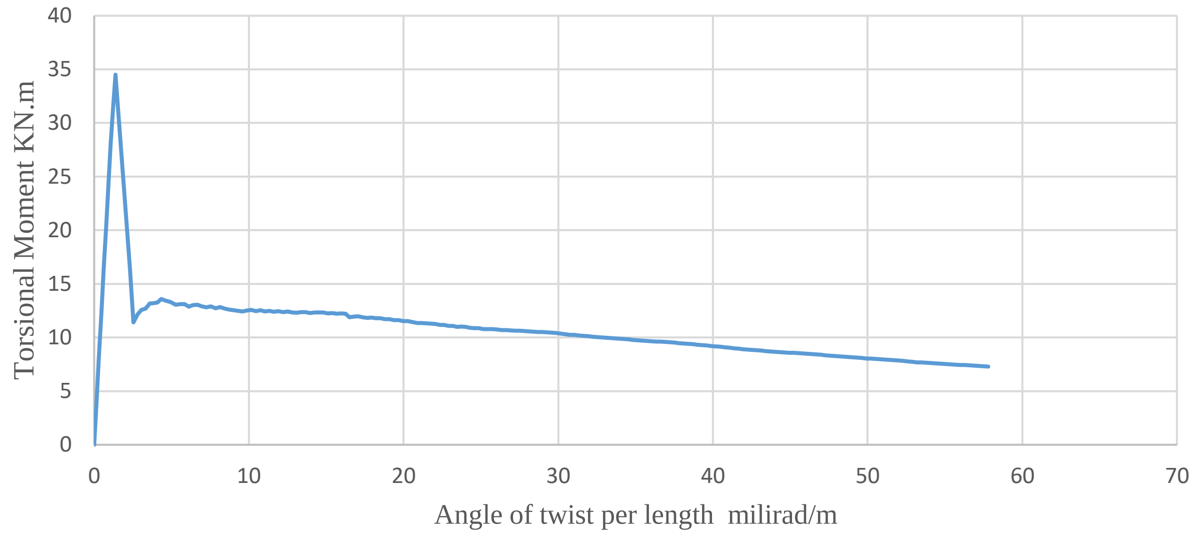


Figure 2-1 Torsional Moment-Angle of twist per length diagram

The shear stress τ is calculated using the following formula based on the assumptions of thin-walled character of the problem.

$$\tau = \frac{M_t}{2At} \quad (1)$$

A is the area enclosed by the cross-section center-line (in this case $= \pi . r^2$), t is the thickness of the experimental tube and r is the radius of the center-line of the cross-section.

The shear strain γ of the middle surface considering no warping of the cross-section is simply calculated by,

$$\gamma = \frac{\partial u}{\partial s} + \frac{\partial v}{\partial x} \quad (2)$$

Where v is the displacement of a cross-section point in the direction of the tangent to the center-line of the cross-section (i.e. in the direction of the co-ordinates S) and u is the warping of the cross-section. Since the cross-section under pure torsion rotates as units about the axis of the tube, it can be written as;

$$v = \emptyset . r \quad (3)$$

And regarding that no warping u at the axisymmetric tube cross-section arises, the shear strain is simply represented as;

$$\gamma = \frac{\partial v}{\partial x} = r\frac{\partial \emptyset}{\partial x} = r\theta \quad (4)$$

By using equations (1) and (4), the stress- strain diagram corresponding to the M_t - θ diagram (Figure 2-1) can be derived as in Figure 2-2.

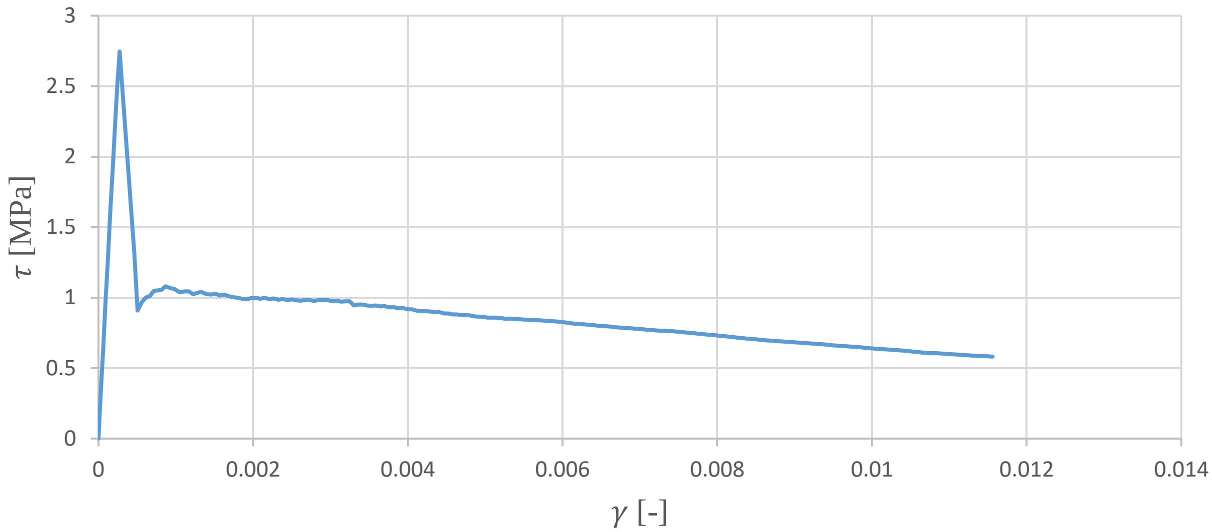


Figure 2-2 Dependence between stress- strain diagram for pure shear mode

3 EXPERIMENTS

This experiment was performed by Václav Ráček et al, in the Faculty of Civil Engineering at Czech Technical University in Prague, in the department of concrete and masonry structures. It was intended to investigate the pure shear performance of fiber reinforced concrete in a pure shear mode without any flexural effect.

3.1 Design of the specimen

The whole specimen is manufactured monolithically. Fiber-concrete specimen consists of the middle part (the tube) and the end parts (cantilevers perpendicular to the tube axis). Cantilevers enable to support the specimen and also to introduce the torque. The specimen is placed in the horizontal position.

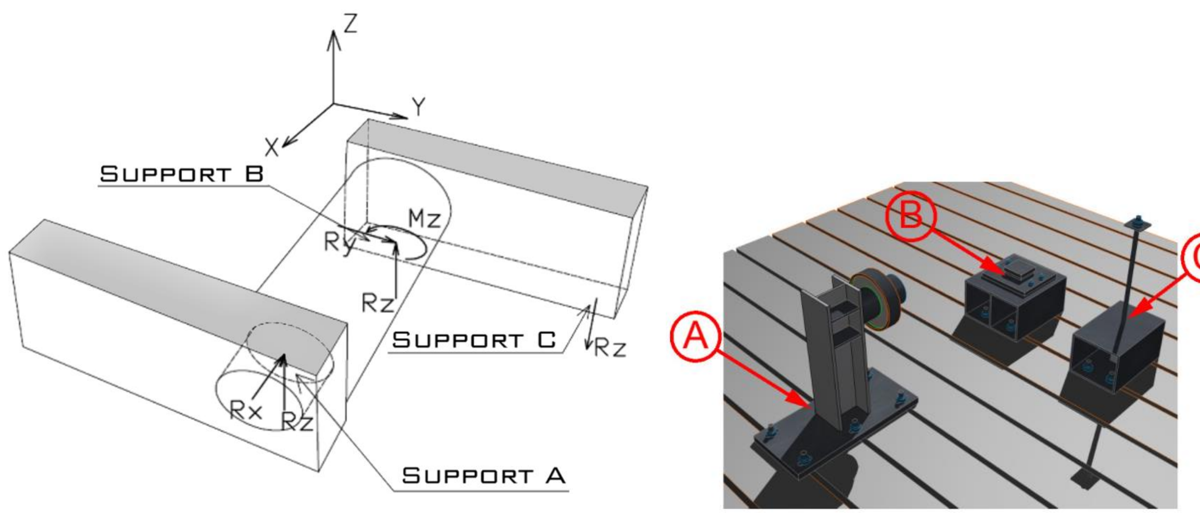


Figure 3-1 Specimen support

A couple of inverse forces on each cantilever are producing a torsional moment in the tube. A force from the hydraulic jack was exerted on one of the cantilevers, and the remaining forces are all reactions which are produced due to the applied supporting conditions. The boundary conditions were arranged in a manner to have a statically determinate structure in order to avoid the formation of restrained torsion which produces additional stress in the tube. In Figure 1, the scheme of specimen including reactions from supports is indicated. Support *A* represents a hinge capable to move in the *y* direction. Support *B* is elastic bearing, which allows *x*-axis displacement. Support *C* is realized by a rod transmitting only vertical tension.

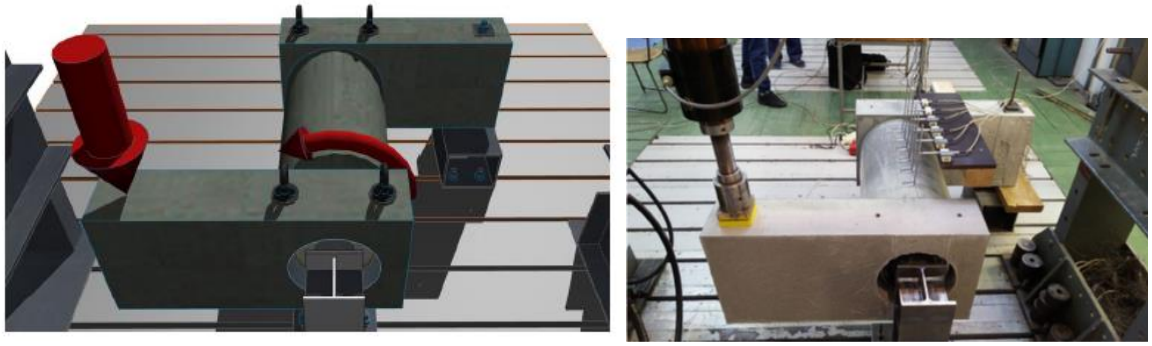


Figure 3-2 Spatial visualization of specimen and its supports

3.2 Production of the specimen

The mould was oriented in vertical position and due to the complexity of the specimen and reinforcement concreting was done in two steps. Firstly, concrete was filled to the bottom cantilever and then from the upper part (look Figure 3-3.).

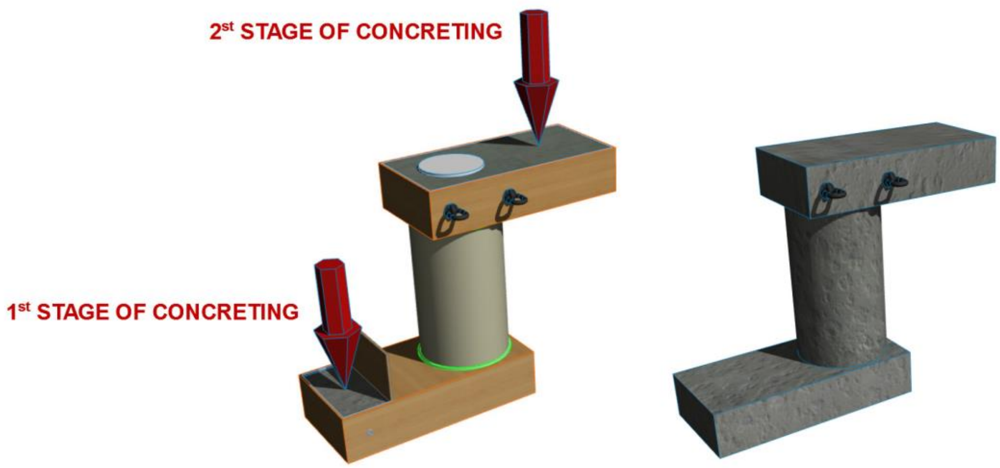


Figure 3-3 Stages of concrete placing

The mixture of concrete was designed to fulfil the requirement of easy workability (nearly self-compacting concrete). During the test, the tube is under homogenous stress in the entire length, therefore the discontinuities in concrete should be avoided for relevant results. Perfect surface was ensured without any cavities (Figure 3-4).

Figure 3-4 Specimen after the removal of the mould

External mould of the tube is made from paper; the internal one is created from polystyrene cylinder (see Figure 3-4). After the concrete solidify, the polystyrene is removed. The newly originated opening enables to put in the support A. Weak point of the designed specimen could be the transition region between cantilevers and a tube. To avoid damaging of concrete in this connection, the transition region was reinforced. These short reinforcing bars securing continuity between the tube and cantilevers differs in their length, so the change of stiffness is more continuous. The specimens were equipped by transport anchors (see Figure 3-4).

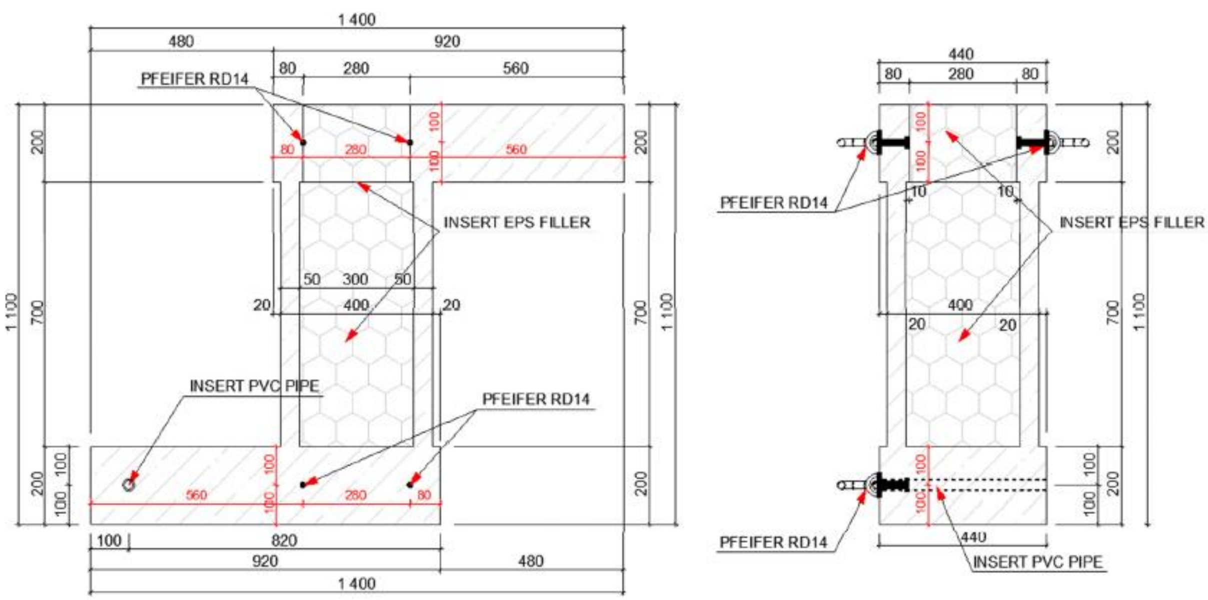


Figure 3-5 Tube drawings (Section through tubes)

3.3 Test procedure

Displacement control method is used for the transfer of the load from the hydraulic jack into the concrete tube through the cantilever. The couple of opposite forces is originated on each side of the tube due to appropriate arrangement of supports. This couple of forces produces the torsion moment in the tube. To avoid damaging of concrete in the connection between the tube and the cantilever, this transition region is reinforced. These short reinforcing bars securing continuity between the tube and cantilevers.

Load is applied by strain increments. The aim is to obtain the relation between the torsional moment and the angle of twist including the post-peak part of the torque-twist diagram. During experimental testing, the displacements of tube surface are monitored by the strain gauges attached to the tube surface (see Figure 3-2). In the initial elastic part of stress-strain diagram the angels of twist between adjacent gauges are constant. When the ultimate shear strength is reached, the crack suddenly propagates through the entire tube. The direction of crack is perpendicular to principal tensile stress, so it is inclined at 45° to tube axis (see Figure 3-6). Then the magnitude of transmitted load immediately decreases. Fibers carry forces in the initiated crack and enable additional strain loading. The final result of the experimental study is the true relation between shear stress and shear strain of fiber-concrete.

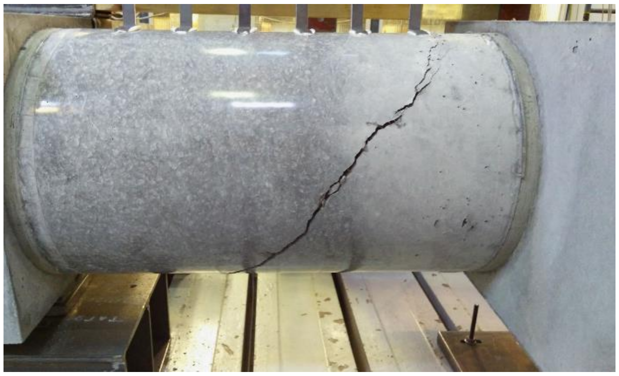


Figure 3-6 Inclined crack

3.4 Experimental results

3.4.1 Stress- strain dependence for shear regime

The loading process is controlled by twist (the angle of twist $\emptyset$) produced by the loading couple and corresponding torque (the torsional moment M_t) is recorded. The angle of twist per unit length $\theta = \frac{\partial \emptyset}{\partial x}$ is calculated and values of the torque M_t as a function of θ can be plotted. Typical shape of the M_t - θ diagram is shown in Figure 3-7.

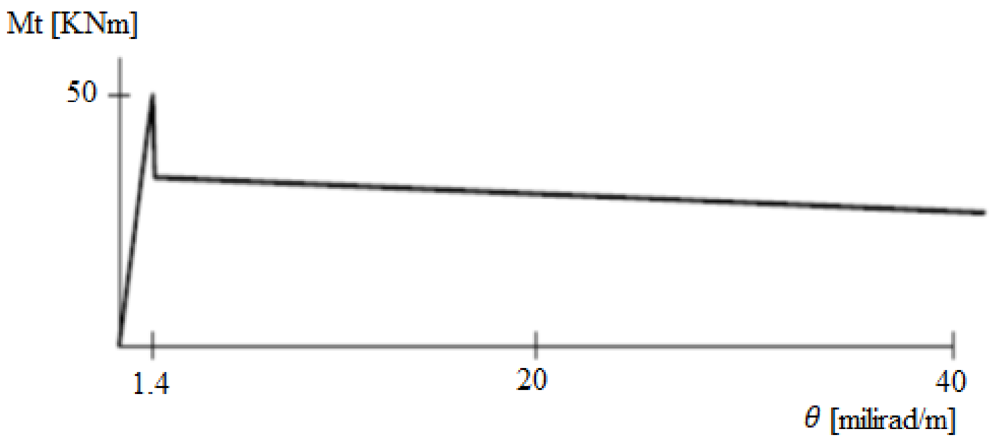


Figure 3-7 Torsional Moment-Angle of twist per length diagram

The shear stress τ is calculated using the following formula based on the assumptions of thin-walled character of the problem.

$$\tau = \frac{M_t}{2At} \qquad (1)$$

A is the area enclosed by the cross-section center-line (in this case A $= \pi . r^2$), t is the thickness of the experimental tube and r is the radius of the center-line of the cross-section.

The shear strain γ of the middle surface considering no warping of the cross-section is simply calculated by,

$$\gamma = \frac{\partial u}{\partial s} + \frac{\partial v}{\partial x} \qquad (2)$$

Where v is the displacement of a cross-section point in the direction of the tangent to the center-line of the cross-section (i.e. in the direction of the co-ordinates s) and u is the warping of the cross-section. Since the cross-section under pure torsion rotates as units about the axis of the tube, it can be written as;

$$v = \emptyset . r \qquad (3)$$

And regarding that no warping u at the axisymmetric tube cross-section arises, the shear strain is simply represented as;

$$\gamma = \frac{\partial v}{\partial x} = r\frac{\partial \emptyset}{\partial x} = r\theta \qquad (4)$$

By using equations (1) and (4), the stress- strain diagram corresponding to the M_t - θ diagram (Figure 3-7) can be derived as in Figure 3-8.

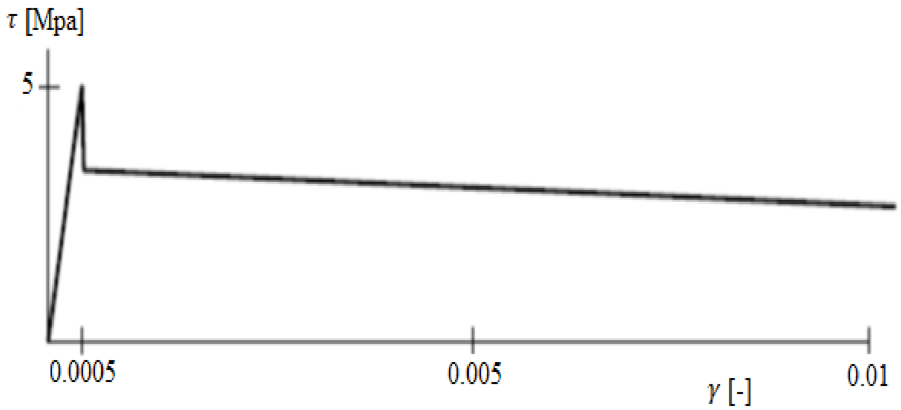


Figure 3-8 Dependence between stress- strain diagram for pure shear mode.

3.4.2 Biaxial stress failure criterion of concrete

Results of the experiments enable to derive the dependency between ultimate strength of different failure modes. Typically, the ratio between ultimate tensile and shear strength can be explicitly obtained. Such knowledge can help to calibrate the biaxial failure criterion of concrete. Recently, there exist several models for description of failure surface. On Figure 3-9 the failure function according to Kupfer is depicted.

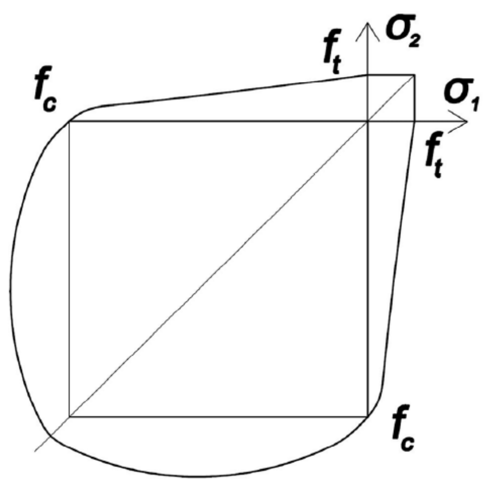


Figure 3-9 Biaxial stress failure criterion for concrete.

3.5 Objectives

The general aim of this thesis is to study the shear resistance of concrete filled steel tubular columns at elevated temperatures. At elevated temperatures concrete and steel separates and most of the load is carried by concrete as steel suffers local buckling. Based on the geometrical and load configuration there are different possible failure modes of concrete. It can fail either in compression, bending or a combination of both including shear. Due to the imperfections and load eccentricity the column maybe subjected to shear or bending. It is generally clear that the tensile strength of a concrete is small compared to compression strength and bending strength. In the meanwhile the shear strength is still less than that of tensile strength. Therefore, a presence of a significant amount of shear force may lead in the shear failure of the concrete column.

In order to study the shear resistance of a fiber reinforced concrete filled steel tubular columns, it is necessary to investigate the pure shear resistance of fiber reinforced concrete, which is the main focus of this thesis. First, in order to investigate the pure shear resistance of FRC, a numerical model is prepared based on the conditions of the experiment. This model will be validated against test result data from the pure shear experiment.

The validated model will be used to carry out a parametric study on the pure shear resistance of FRC at elevated temperatures. This will be very useful in order to study the shear resistance of CFSTC at elevated temperatures.

Also, in order to understand the fiber reinforced concrete material behavior, failure modes and the size effect, numerical models of a cube, cuboid and circular column are prepared and studied.

The primary goal for this dissertation is to improve the existing knowledge on shear resistance of CFSTC at elevated temperatures. The following are the specific objectives;

- Creation of 3-Dimensional numerical model of fiber reinforced concrete cylinder using ATENA software according to the experimental data parameters;
- Validation and verification of numerical model by comparing the numerical outputs against the experimental results;
- Perform sensitivity tests on numerical model in order to understand the pure shear resistance of fiber reinforced concrete at elevated temperatures;
- Investigate the tensile and compressive behavior of fiber reinforced concrete and its failure modes considering the size effect using numerical models of cube, cuboid and a full size column;
- Examine the analyses results in order to summarize the findings and enrich the existing knowledge of shear resistance of fiber reinforced CFSTC.

4 NUMERICAL MODELING

With the latest technological discoveries and the advanced finite element software's it is possible to simulate the response of almost of all complex structural components. However, there are still some difficulties in using these numerical analysis as it requires special knowledge to represent accurately the actual phenomenon using the appropriate finite element model with the lowest computational cost. Choice of the mesh, node number, integration point number through the element thickness and time-step size for constitutive law integration depend upon resources, geometry, type of loading and required accuracy.

The software ATENA was used in the present study to model the pure shear model of fiber reinforced concrete. A 3-dimensional model of fiber reinforced concrete cylinder was modelled for the study of the pure shear strength. In addition to that ATENA was used to model a cube, cuboid and full column size in order to study the material properties and behavior of fiber reinforced concrete. This software package is mainly specialized and developed for the computation of concrete structures. The 3-Dimensional model used for the pure shear properties of fiber reinforced concrete was validated against the experimental test which was conducted in Czech Technical University in Prague, in Department of Concrete and Masonry Structures by Václav Ráček et al.

4.1 Material properties

The properties of the fiber reinforced concrete which were used in the model were determined by an experimental test within the framework of "Models of Fiber reinforced concrete columns in case of fire". The amount of steel fibers used in the fiber concrete mix was 1.7% by mass. Experimental tests were performed on conventional bodies at normal temperatures and elevated temperatures of 200 ° C, 400 ° C and 600 ° C. On the basis of the acquired knowledge a material model was made, which is to serve for numerical simulations of composite steel columns on the spatial model. The input material properties of FRC which are used in ATENA software at different temperatures are given in Table 4-1.

Table 4-1 Material properties of Fiber reinforced concrete at different temperatures

Parameter		Properties at(20°C)	Properties at(200°C)	Properties at(400°C)	Properties at(600°C)
Material Model					
Basic	Young modulus [MPa]	36000	34000	14000	4500
	Poisson ratio [-]	0.2	0.2	0.2	0.2
	Tension strength [MPa]	4.4	3.8	3.6	1.4
	Compression strength [Mpa]	-62	-57	-52	-29
Tensile	Fracture energy [KN/m]	0.103	0.103	0.103	0.02
	Fixed crack	1	1	1	1
	Activate crack spacing	No	No	No	No
	Activate tension stiffening	No	No	No	No
	Activate aggregate interlock	0.02 m	0.02 m	0.02 m	0.02 m
	Activate shear factor	No	No	No	No
	Activate unloading factor	No	No	No	No
Compressive	Plastic strain- EPS CP	-0.0009	-0.0009	-0.0011	-0.0012
	Onset of crushing – FCO [MPa]	-9.21	-9.21	-9.21	-9.21
	Critical Comp disp-WD [mm]	-0.15	-0.17	-0.3	-0.35
	Fc reduction	0.8	0.8	0.8	0.8
Miscellaneous	Excentricity-EXC	0.51	0.51	0.51	0.51
	Direc of pl flow-BETA	0	0	0	0
	Rho density [kg/m^3]	2300	2300	2300	2300
	Thermal expansion-alpha [C^{-2}]	0.000012	0.000012	0.000012	0.000012
Element geometry	Geometrical non linearity	Linear	Linear	Linear	Linear
	Idealisation	3D	3D	3D	3D
	Non-quadratic element	No	No	No	No

4.2 Numerical model for pure shear properties of FRC

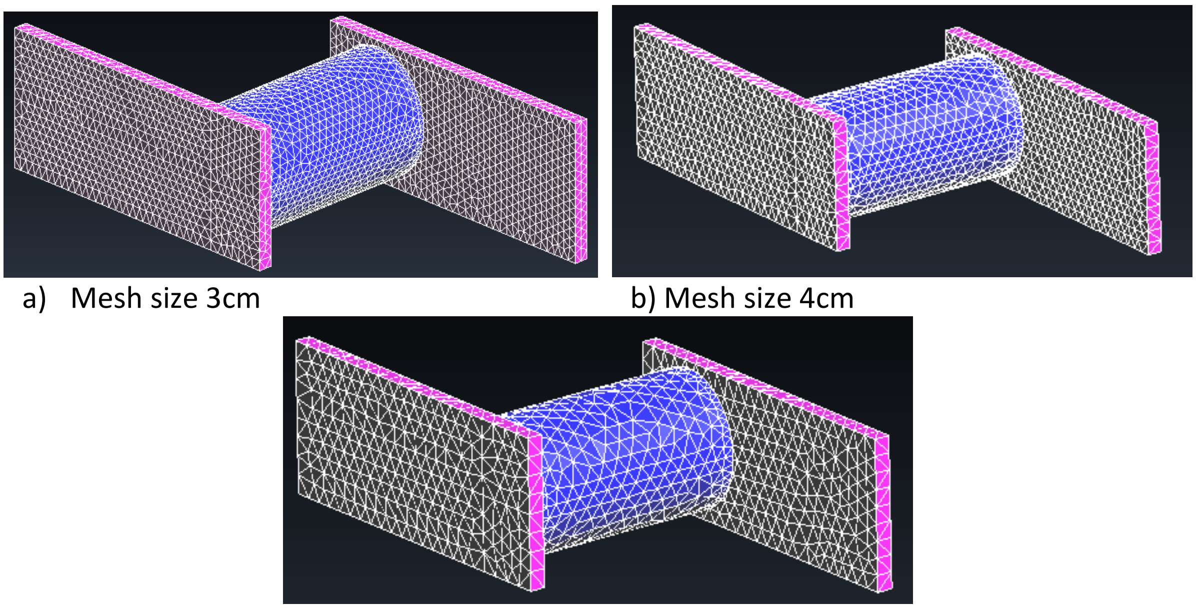
4.2.1 Meshing

Meshing plays a crucial role in the analysis of finite element modelling. A finer finite element mesh usually gives better calculation results. However, as a mesh is made finer, the computation time and cost increases. A finer mesh depends on many factors. Among them is the cost versus the accuracy to receive.

The mesh density required can be a function of many factors. Among them are the stress gradients, the type of loadings, the boundary conditions, the element types used, the element shapes, and the degree of accuracy desired.

The type of the mesh elements (quadratic or triangular) are also an important aspect to take into account for the refinement process, being necessary for such process, to have in consideration the shape of the mesh for the structural elements (undistorted or distorted). It is important to keep the elements with an appropriate aspect ratio.

For this study a tetrahedral type of meshing was chosen with an approximate element size of 4cm.This size was selected after doing different mesh size convergence study in order to give more accurate result and relatively with less computational cost.



a) Mesh size 3cm

b) Mesh size 4cm

C) Mesh size 5cm

Figure 4-1 Different mesh sizes of the Numerical Model

4.2.2 Support and loading condition

Support and load conditions were applied to the FE models according to the experimental test. Load was applied at the top of one of the cantilever arm. The supporting conditions on the model were also made in similar arrangement which recreate the test conditions. The master- slave fixed contact surface was used to connect the tube concrete and the steel cantilever.

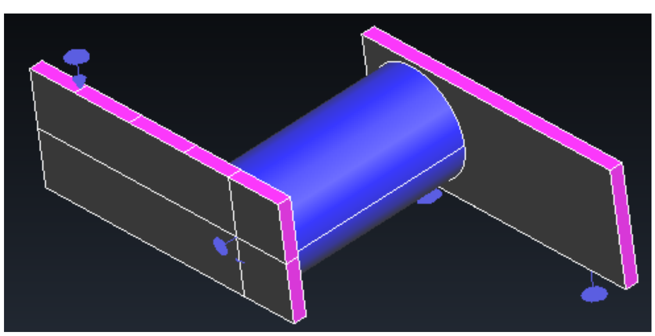


Figure 4-2 Support and loading conditions of the model

4.2.3 Validation

The validation process was performed through the comparison between the numerical analysis and the experimental data. Influence of the element size, mesh convergence study, the formation of the shear crack obtained from the experimental test was investigated as part of the validation. Results of the analyses obtained from the numerical model were similar to the experimental results.

4.2.3.1 *Convergence Study*

To verify the convergence of FE results, 3 different mesh densities were performed. The convergence studies were carried out to obtain the optimum finite element mesh size. The finely meshed model intends to better capture the stress capacity before cracking and post cracking.

By comparing the 3 different meshes, it is possible to obtain a clear analysis of the accuracy of the model. Figure 4-3 shows the stress-strain curve for the three different meshes.

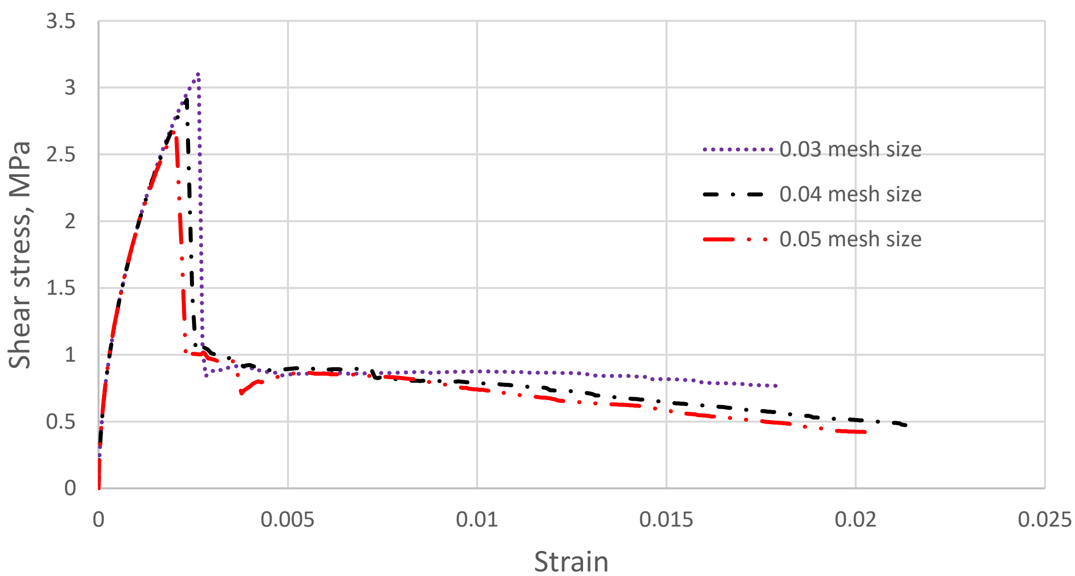
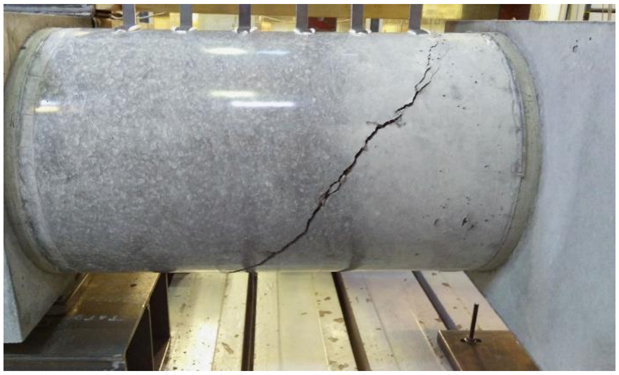
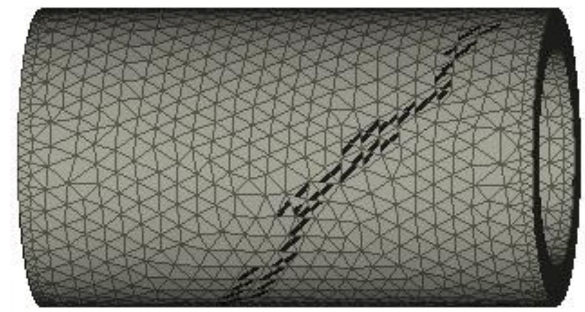


Figure 4-3 stress-strain curves

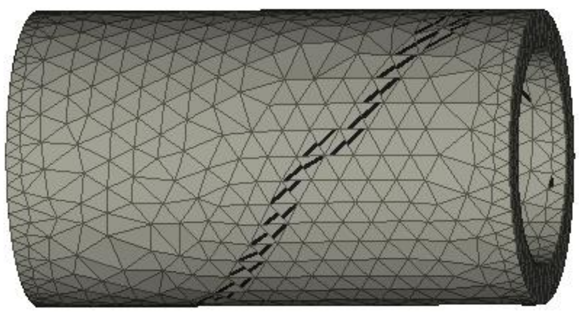
The formation of the diagonal crack for the three mesh sizes are also compared to the experimental results. The crack obtained from the finer mesh represents more accurately the experimental one.



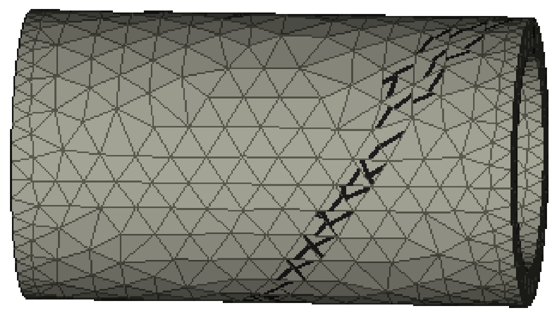
a) Crack experiment



b) Crack 0.03 mesh size



c)Crack 0.04 mesh size



d) Crack 0.05 mesh size

Figure 4-4 Diagonal shear crack

4.2.4 Parametric study

Parametric study is a desired element of the experimental work and an essential part of the numerical analysis. The cost and time needed to perform a number of experimental study in order to investigate the response behavior of different parameters is very high, but these difficulties can be reduced with the help of numerical analysis significantly. The validated model can be used to investigate the structural behavior by conducting a sensitivity study on different parameters.

As the main interest of this dissertation is the shear resistance at elevated temperature, a sensitivity study is done by choosing different temperatures of concrete. Material input characteristics of fiber reinforced concrete at elevated temperatures of 200℃, 400℃ and 600℃ were already studied.Therefore, these temperatures were used to study the pure shear characteristics of fiber reinforced concrete.

To investigate the mechanical properties of fiber reinforced concrete at elevated temperature in case of pure shear, it was necessary to rely on the numerical model which was verified against the experimental result at ambient temperature, because it was not possible to find experiments conducted at elevated temperature for the shear resistance of fiber reinforced concrete. Using the numerical model, shear characteristics of fiber reinforced concrete at temperatures of 200℃, 400℃ and 600℃ was studied. In order to compare the results of the shear strength at elevated temperature, the percentage decrease of the shear strength was checked against the experimentally obtained percentage decrease of compressive and tensile strength of the fiber reinforced concrete at each given temperatures of 200℃, 400℃ and 600℃. The results are provided in Table 5-2. Looking on the results, the percentage decrease of the shear strengths lies close to the range of the percentage decrease of compressive and tensile decrease. Therefore, it was concluded that the results of shear strength from the numerical experiment was genuine.

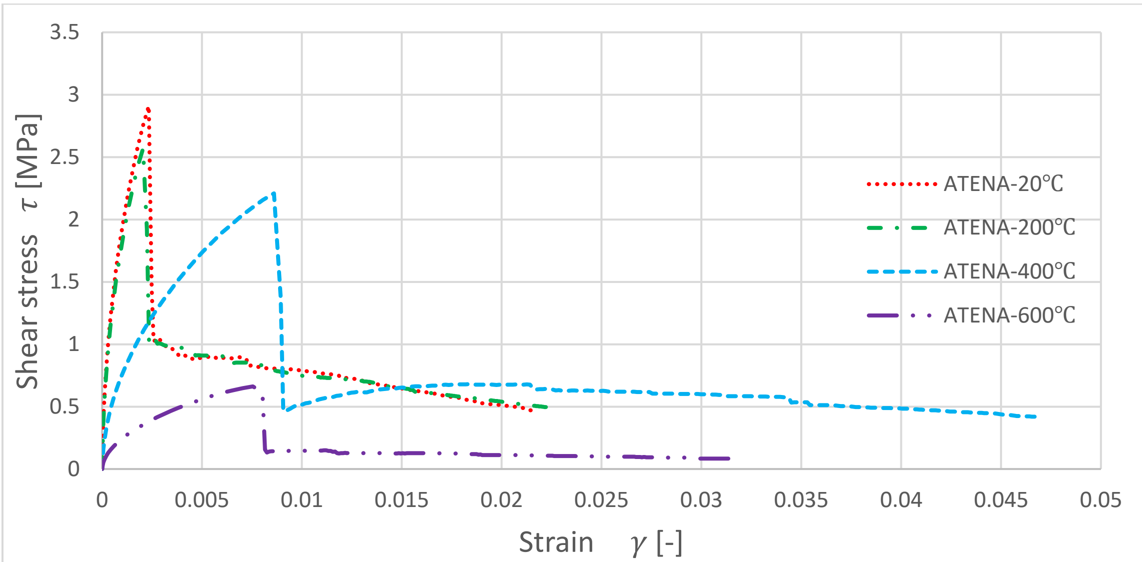


Figure 4-5 Pure shear stress τ – Strain γ diagram for SFRC at elevated temperatures

Table 4-2 Comparison of Compressive strength, Tensile strength and Shear strength

	Compressive strength		Tensile strength		Shear strength	
Temp. (℃)	Ult. Stress [MPa]	% decrease	Ult. Stress [MPa]	% decrease	Ult. Stress [MPa]	% decrease
20	62	0.0	4.4	0.0	2.91	0.0
200	57	-8.1	3.8	-13.6	2.57	-11.8
400	52	-16.1	3.6	-18.2	2.21	-24.1
600	29	-53.2	1.4	-68.2	0.66	-77.3

4.3 Numerical model of a Cube

4.3.1 General

It is very essential and useful to study the material properties of FRC on different sizes of elements. It is interesting to know how these properties are affected when we consider different object sizes. In order to achieve these study three numerical models of FRC of different sizes, a cube, a cuboid and a column were modeled in ATENA FE software. The material properties of the FRC which is used in this numerical models is obtained from the experiment, see Table 4-1. Also a comparison study is made among normal concrete and FRC.

Firstly, a numerical study was done on a cube of FRC. The compressive and tensile strength of the FRC cube were obtained both at ambient and elevated temperatures numerically.

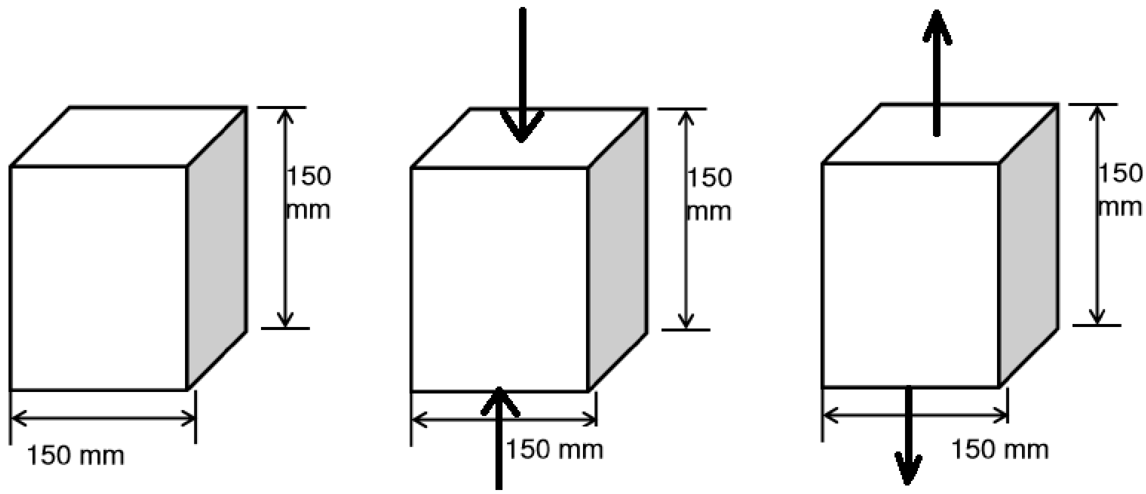


Figure 4-6 FRC cube compressive and tensile tests

4.3.2 Compressive and tensile characteristics of FRC cube

From the stress-strain curves in Figure 4-7 and Figure 4-8 it is observed that the compressive strength of FRC decreases by around 24% at 400℃ and by 58% at 600℃, while with only 5% decrease at 200℃. From figure 4-7 it can be depicted that the initial stiffness of FRC for both compression and tension is almost the same till 200℃ and it reduces significantly with further increase of temperature. The comparison is given in table 4-3.

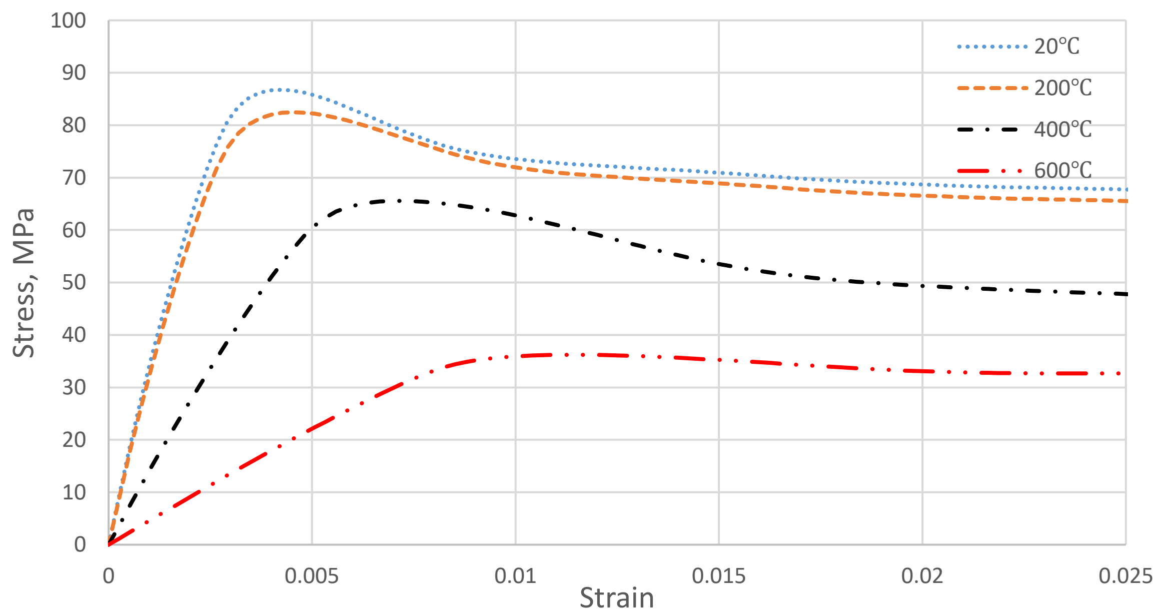


Figure 4-7 Compressive stress-strain curve of a FRC cube at different elevated temperatures

The FRC suffers more loss in tensile strength at higher temperatures comparing to the compressive strength. The post crack behavior shows that the steel fibers present in the concrete enables it to sustain a considerable amount of stress at temperatures lower than 200℃, but with the increase of temperature the tensile strength is reduced significantly and also the post crack load carrying capacity of the cube is very limited.

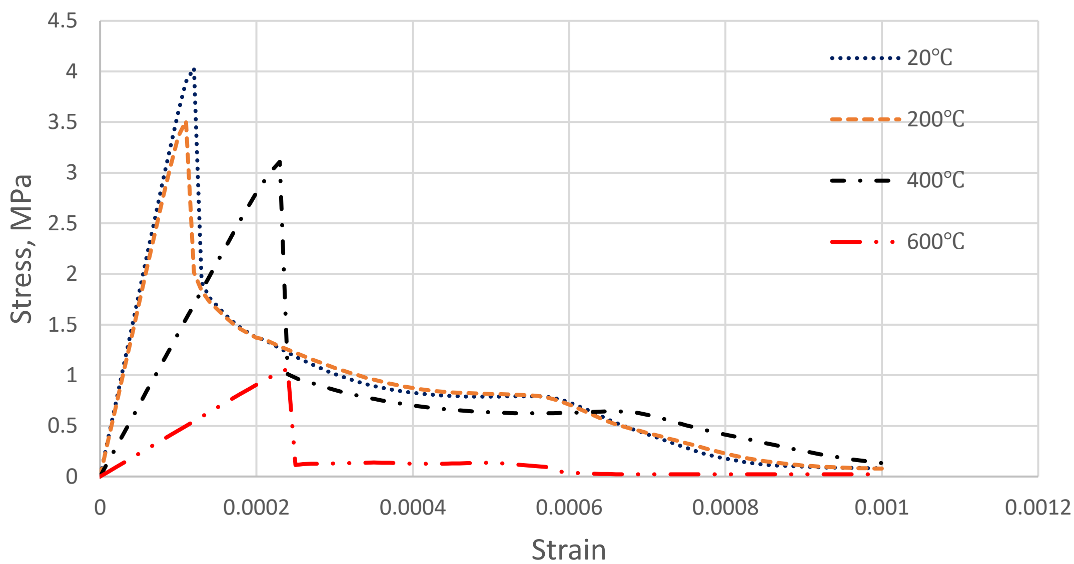


Figure 4-8 Tensile stress-strain curve of a FRC cube at different elevated temperatures

Table 4-3 Comparison of compressive and tensile strength of a FRC cube at elevated temperatures

Item	Temperature, ℃	Compressive Strength (MPa)		Tensile Strength (MPa)		% of Tensile strength to Compressive strength
		Max.	% decrease	Max.	% decrease	
Cube 150x150x150mm	20	86.76	0.0	4.03	0.0	4.65
	200	82.48	-4.9	3.51	-12.9	4.26
	400	65.58	-24.4	3.11	-22.9	4.74
	600	36.20	-58.3	1.06	-73.7	2.93

4.3.3 Compressive and tensile characteristics of a cube of different materials

Also, in order to compare different materials, three materials were studied. Normal concrete, and other two types of fiber reinforced concrete. The properties of the first fiber reinforced concrete (FRC1) was taken from Table 4-1, which was obtained from an experiment and the second type of fiber reinforced concrete (FRC2) was generated from the ATENA FE software using the cementitious2 type material which has similar properties as FRC. The compressive and tensile properties of the three materials used is given in table below.

Table 4-4 mechanical properties of different materials

Type of concrete	Compressive strength, (MPa)	Tensile strength , (MPa)
Normal Concrete (EC2)	48	3.50
FRC1	62	4.40
FRC2	69.2	4.39

One of the advantages of using fiber reinforced concrete is that it significantly increases the ductility of the concrete. The normal concrete is more brittle than the fiber reinforced concrete. This can be observed clearly on the uniaxial compression stress-strain diagram in Figure 4-9.

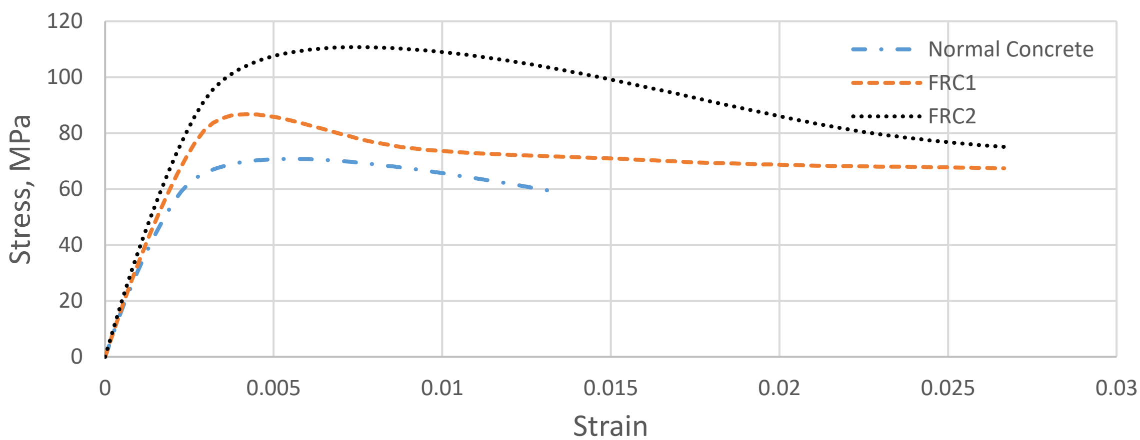


Figure 4-9 Compressive stress-strain curve of FRC different materials at ambient temperature

Concerning, the tensile behavior of the materials, it can be shown that the FRC2 has shown more post crack stiffness comparing to the normal concrete and FRC1.

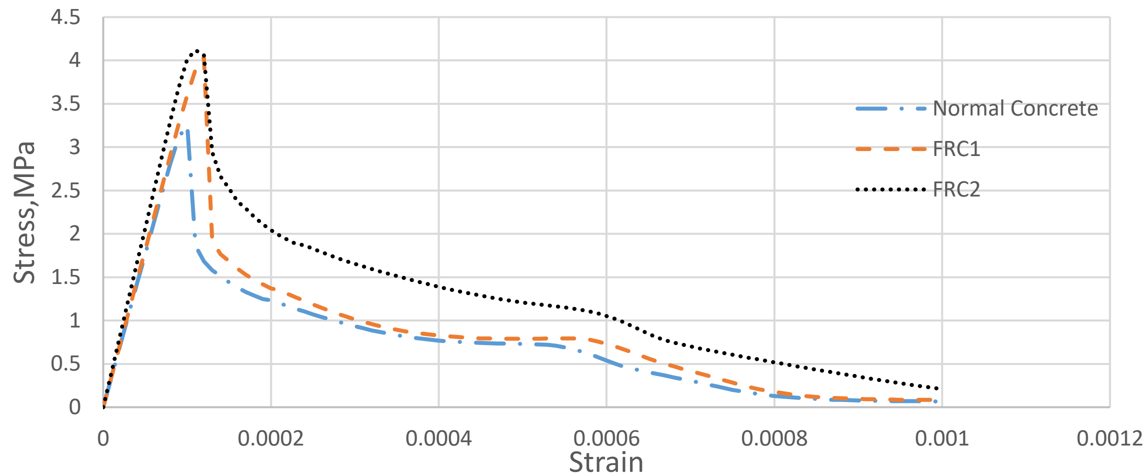


Figure 4-10 Tensile stress-strain curve of different concrete materials at ambient temperatures

4.3.4 Failure modes of a cube

The failure of fiber reinforced concrete under uniaxial loading is affected by the loading environment and the end conditions. It is observed that cracks and micro cracks are formed on the surface of the cube in the direction of loading, and these cracks leads in the stress softening.

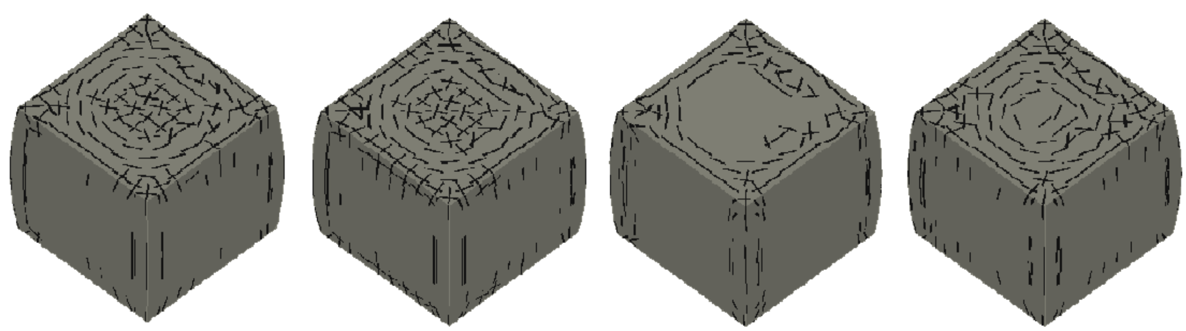


Figure 4-11 Uniaxial compression failures of FRC cube at different temperatures

Under uniaxial tension, propagation of cracks and micro cracks takes place along a plane normal to the loading direction and these leads to strain softening.

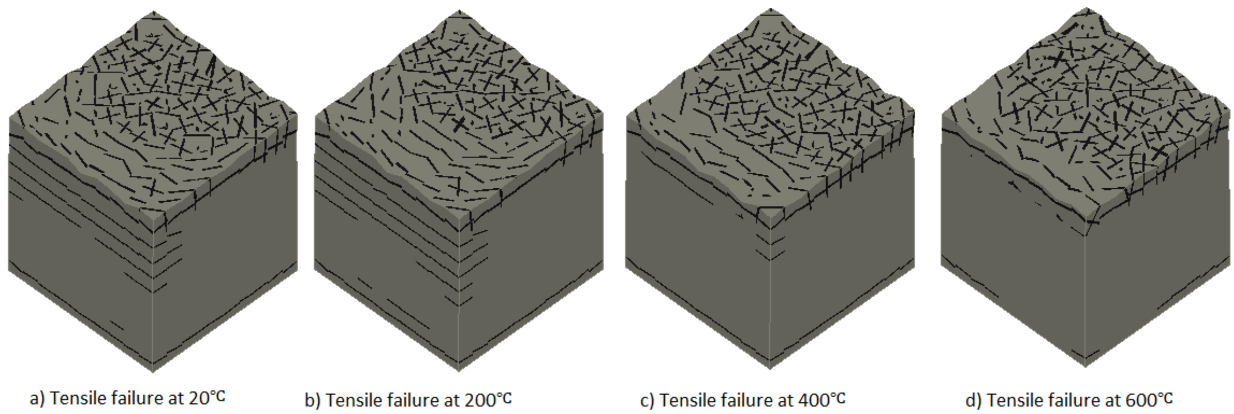


Figure 4-12 Uniaxial tension failures of FRC cube at different temperatures

4.4 Numerical model of a Cuboid

4.4.1 General

In order to study the effects of size to the compressive and tensile characteristics of a fiber reinforced concrete, a cuboid with dimensions 150x150x300 mm was modeled numerically in addition to the cube. The material properties used both in the cube and cuboid were the same and a comparison of the result is made both in compression and tension.

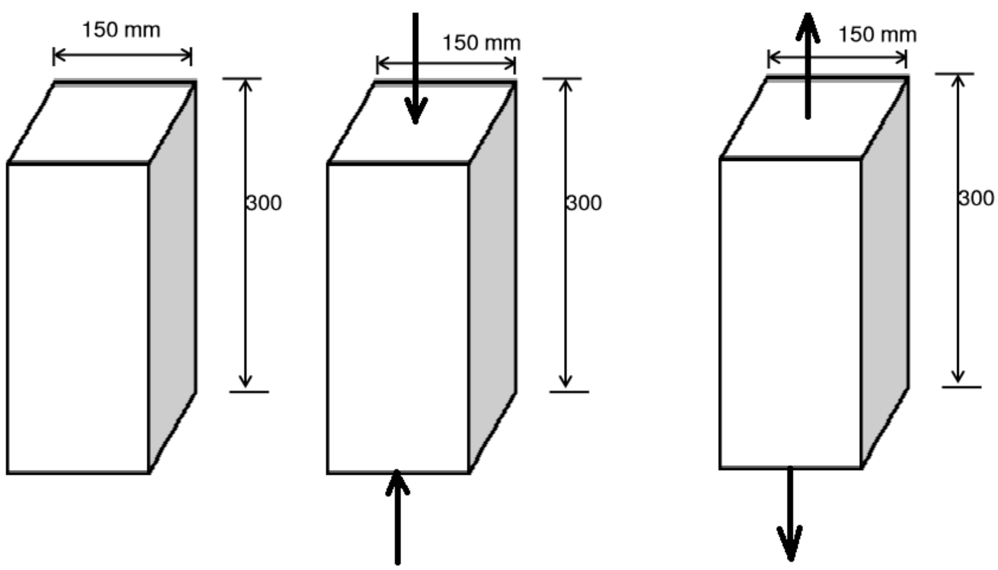


Figure 4-13 Cuboid compressive and tensile tests

4.4.2 Compressive and tensile characteristics of FRC cuboid

A softening behavior of FRC is observed in the compressive stress-strain curve of the cuboid. The variation of the compressive strength with the increase of temperature is similar to that obtained in compression properties of the FRC cube, but generally the compressive strength is reduced, it is explained under the size effect in section 4.4.5.

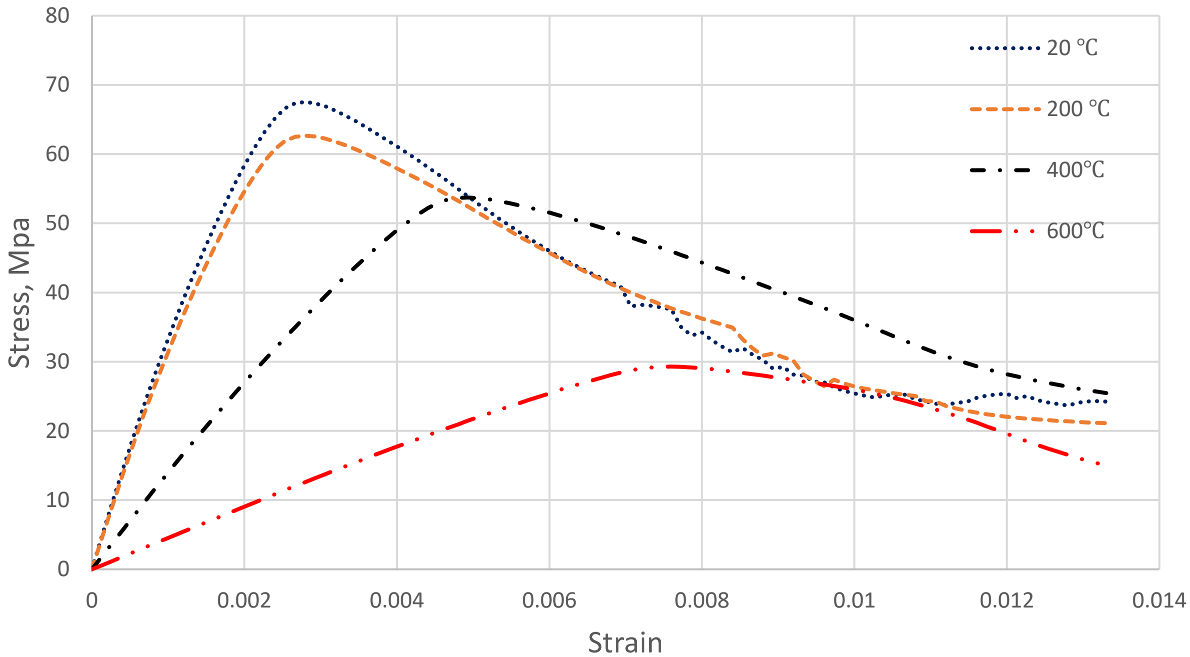


Figure 4-14 Compressive stress-strain curve of a FRC cuboid at different elevated temperatures

The tensile strength of FRC cuboid decreases with the increase of temperature. The change is relatively small up to temperatures of 200℃ and losses approximately 23% and 74% for temperatures of 400℃ and 600℃ respectively (see table 4-5).

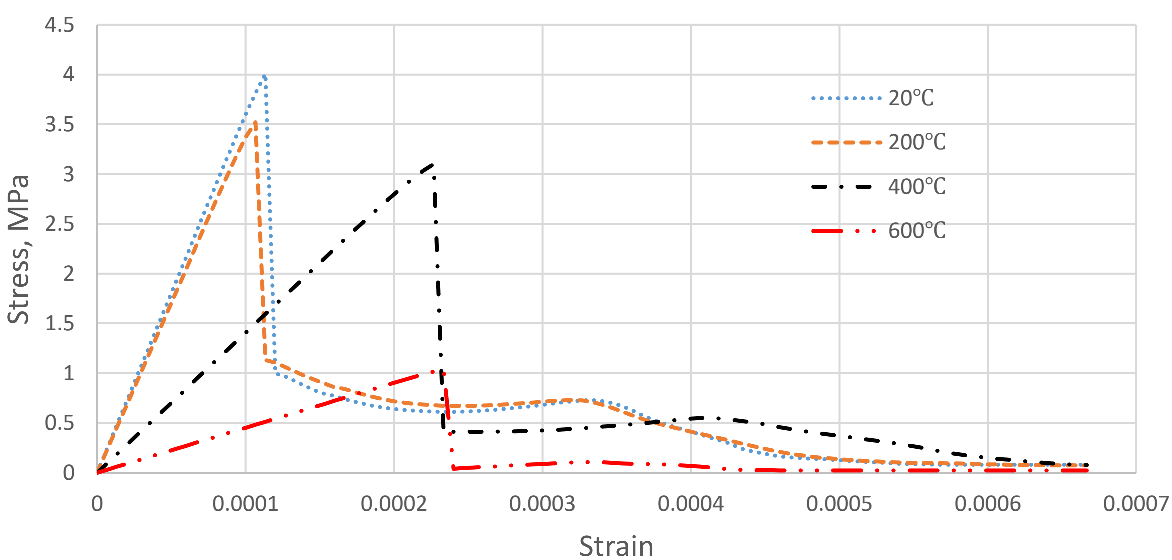


Figure 4-15 Tensile stress-strain curve of a FRC cuboid at different elevated temperatures

Table 4-5 Comparison of compressive and tensile strength of a FRC cuboid

Item	Temperature, °C	Compressive Strength (MPa)		Tensile Strength (MPa)		% of Tensile strength to Compressive strength
		Max.	% decrease	Max.	% decrease	
Cuboid 150x150x300mm	20	67.53	0	4.01	0.00	5.93
	200	62.65	-7.23	3.52	-12.14	5.62
	400	53.73	-20.45	3.10	-22.63	5.77
	600	29.30	-56.62	1.04	-74.03	3.55

4.4.3 Compressive and tensile characteristics of a cuboid for different materials

In order to compare the material characteristics of a cuboid for different materials, three materials were selected, one normal concrete and two fiber reinforced concretes. The properties of these materials are the same as those used in a cube explained in section 5.3.3.

The compressive and tensile stress-strain cures of the three materials are given in figure 4-16 and figure 4-17.

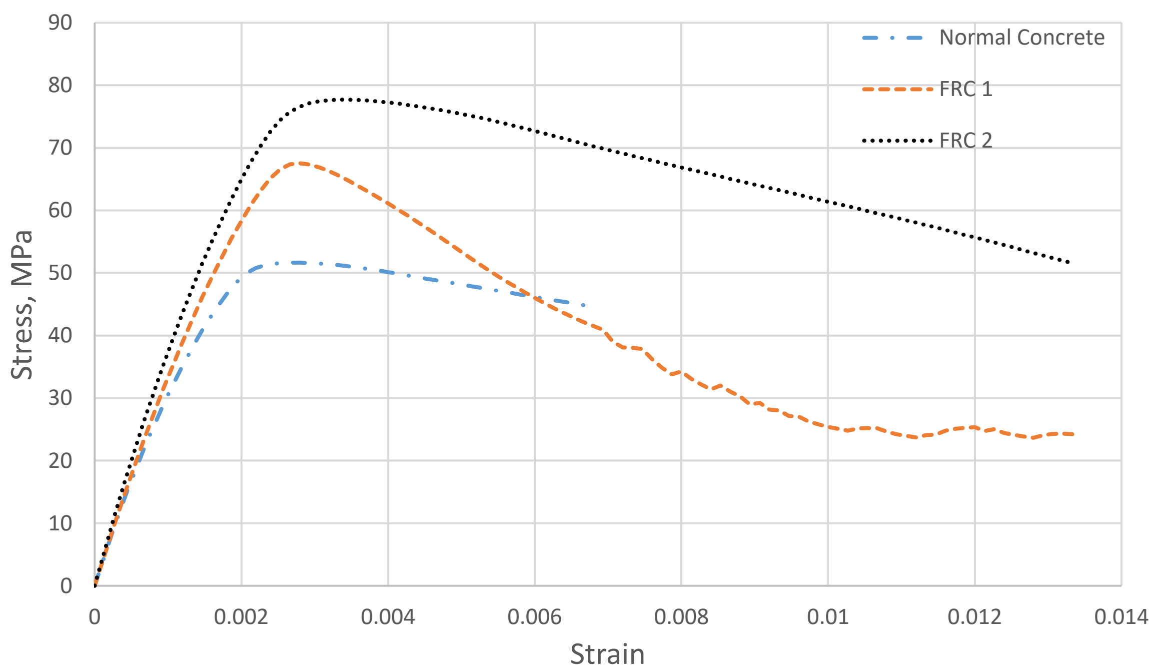


Figure 4-16 Compressive stress-strain curve of different concrete at ambient temperature

Concerning, the tensile behavior of the materials, it can be shown that the FRC2 has shown more post crack stiffness comparing to the normal concrete and FRC1.

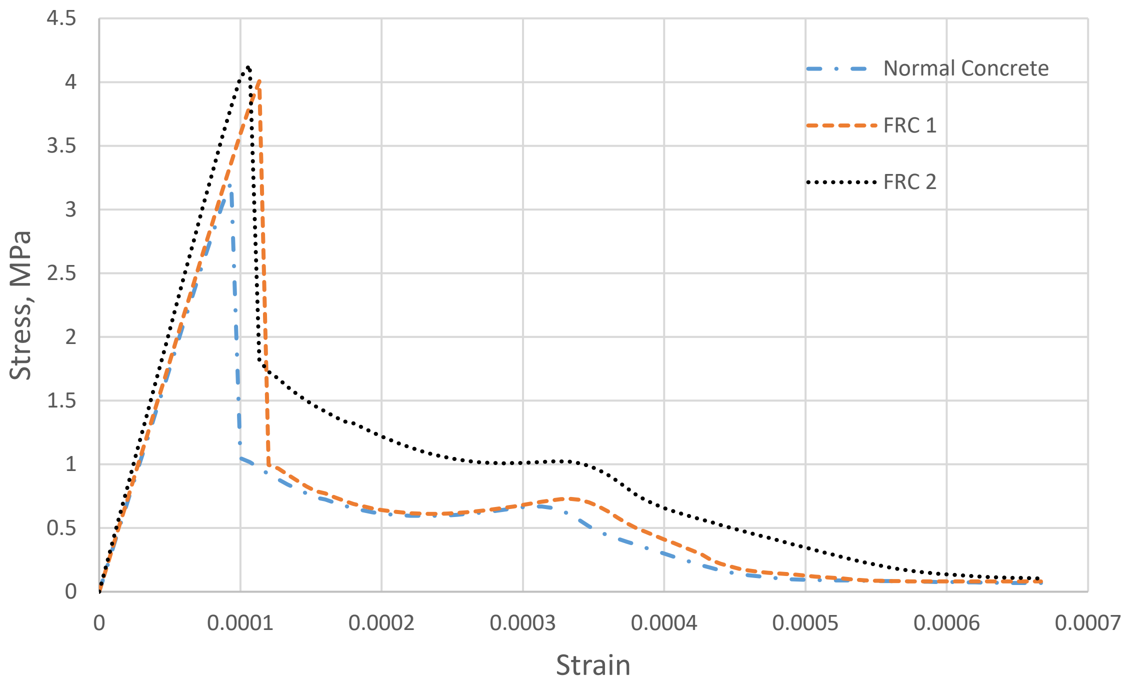
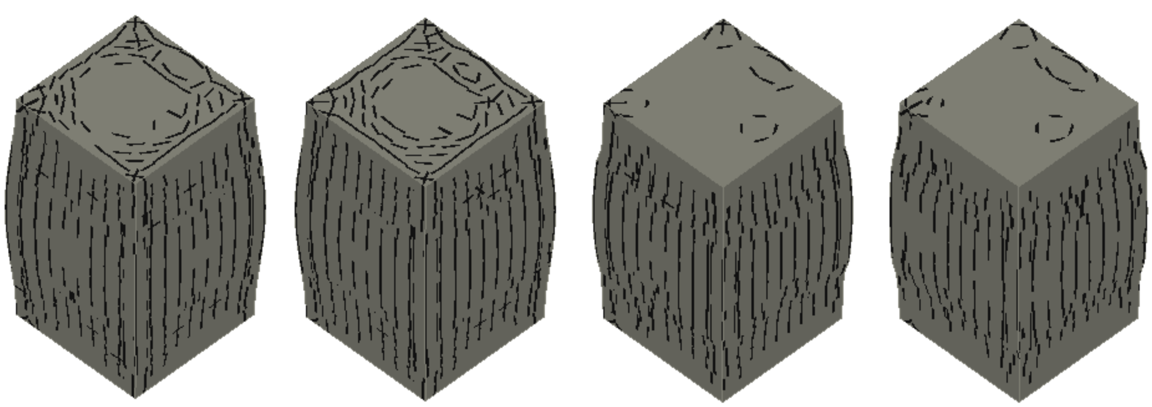


Figure 4-17 Tensile stress-strain curve of different concrete materials at ambient temperatures

4.4.4 Failure modes of a cuboid

In uniaxial compression cracks are formed on the surface of the cuboid along the direction of the application of the load. More number of cracks are observed on the cuboids at temperatures of 20℃ and 200℃ comparing to those at 400℃ and 600℃. The reason for this is the increased ductility of FRC at elevated temperatures and the reduced load carrying capacity.



a) Compression failure at 20℃ b) Compression failure at 200℃ c) Compression failure at 400℃ d) Compression failure at 600℃

Figure 4-18 Uniaxial compression failures of FRC cube at different temperatures

Under uniaxial tension, propagation of cracks and micro cracks takes place along a plane normal to the loading direction and these leads to strain softening.

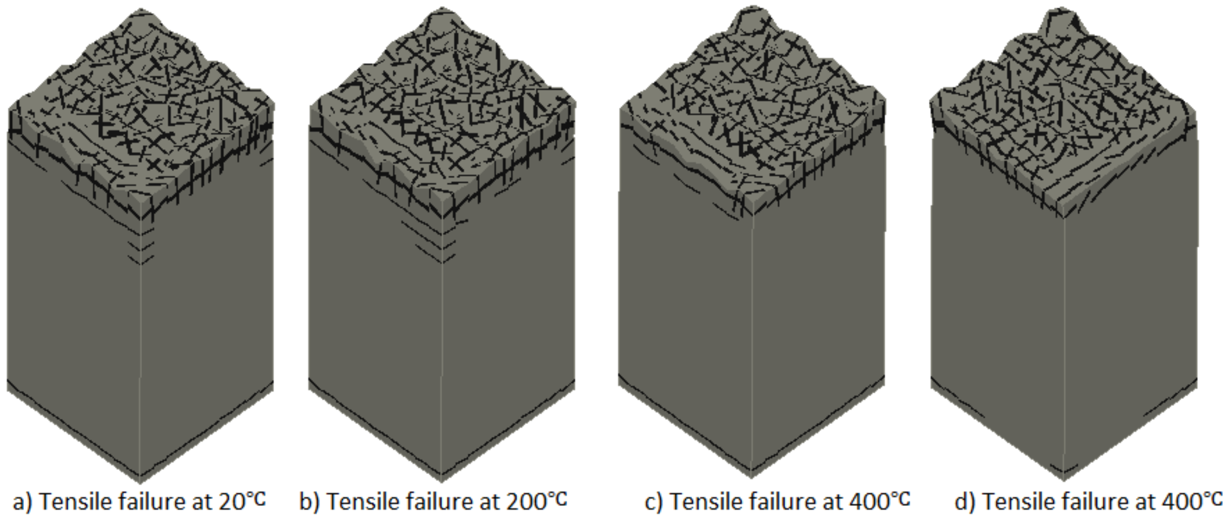


Figure 4-19 Uniaxial tension failures of FRC cube at different temperatures

4.4.5 Size effect of FRC material

A comparison of the compressive strength and tensile strength for the FRC cube and FRC cuboid were made in order to study the size effect on the characteristic behavior of the material. A variation of compressive nominal strength with characteristic size of a structural member was observed. The compressive strength of the cube is about 20% higher than the compressive strength of the cuboid made from the same material. Also the cuboid experiences a pronounced stress softening comparing to the cube (see figure 4- 20).

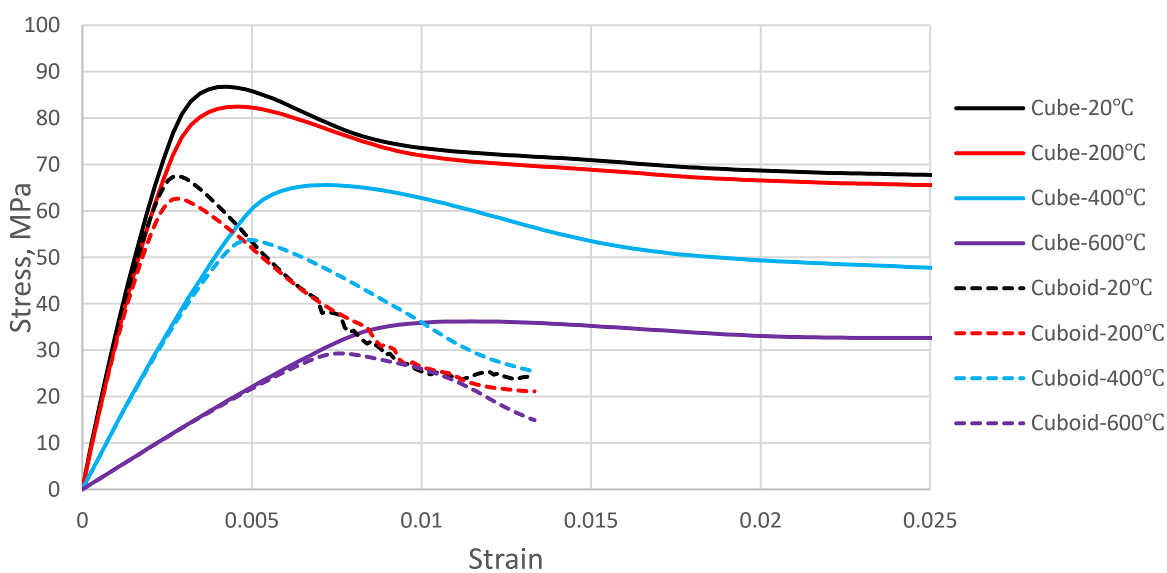


Figure 4-20 Comparison of compressive strength of a FRC cube and a FRC cuboid.

Table 4-6 Comparison of compressive strength of a FRC cube and a FRC cuboid.

Temperature, ℃	Compressive strength, MPa		% change of the cuboid relative to the cube.
	Cube	Cuboid	
20	86.76	67.53	22.16
200	82.48	62.65	24.03
400	65.58	53.73	18.07
600	36.20	29.30	19.08

Unlike the compressive strength, the tensile strength of the cube and cuboid are almost the same. Therefore it can be concluded that the effect of size is observed with the reduction in compressive strength while it has only a very small effect on the tensile strength.

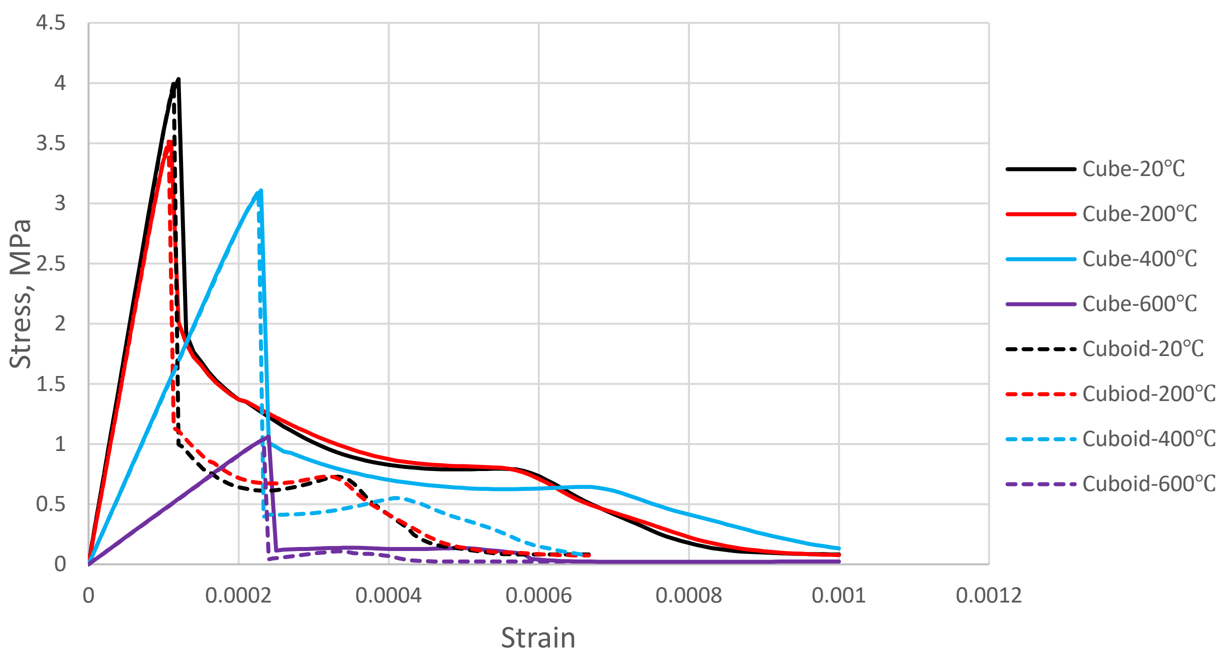


Figure 4-21 Comparison of tensile strength of a FRC cube and a FRC cuboid

Table 4-7 Comparison of tensile strength of a FRC cube and a FRC cuboid

Temperature, ℃	Tensile strength, MPa		% change
	Cube	Cuboid	
20	4.03	4.01	0.65
200	3.51	3.52	-0.24
400	3.11	3.10	0.26
600	1.06	1.04s	1.96

4.5 Numerical model of FRC Column

4.5.1 Geometry and finite element mesh

A three dimensional model for simulating the behavior of FRC circular column was developed with ATENA finite element software. It consists of two parts, the concrete and the loading plate. It has 2000 mm length and 310 mm diameter.

The loading plate was modelled as a perfectly elastic part and through this element the axial load was transmitted to the concrete column.

A parametric study was made considering the eccentricity of the application of the load and different elevated temperatures. The material input properties of the FRC used in modelling were obtained experimentally and indicated in Table 4-1. The model was meshed by a three dimensional tetrahedral mesh both for the column and the loading plate.

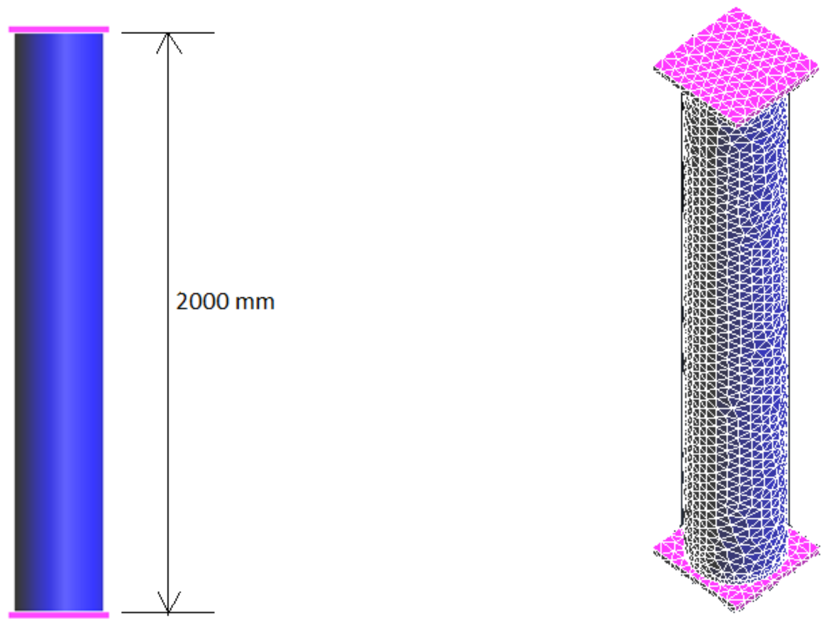


Figure 4-22 FRC column ATENA model

4.5.2 Parametric study

The main objective of the parametric study is to investigate the effects of changes of some specific parameters in the numerical model. A range of the parameters are selected according to the functionality and which can bring some changes in the behavior of the structure. The parameters considered in this study are the eccentricity of the application of the load at different elevated temperatures.

2.3.1.1 Sensitivity on eccentricity of the load

The axial load could be applied either without eccentricity or with eccentricity and it has a great role on the behavioral changes of the column. The maximum compressive strength and failure mode of the column is significantly affected due to the eccentricity of the load. For the parametric study eccentricities of e = 0 mm, e=10 mm, e=15 mm and e= 30 mm were considered. Temperature was kept constant in order to see the real significance of the eccentricity. The maximum load capacity of the column for each eccentricity is compared against each other and it is given through figures 4-23 to 4-26.

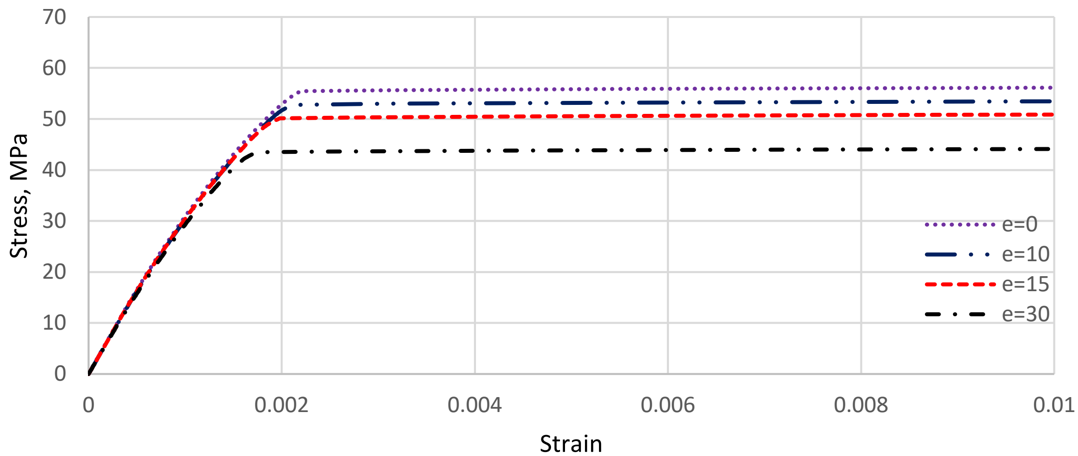


Figure 4-23 Effect of eccentricity of the load on compressive strength of a FRC column at 20℃

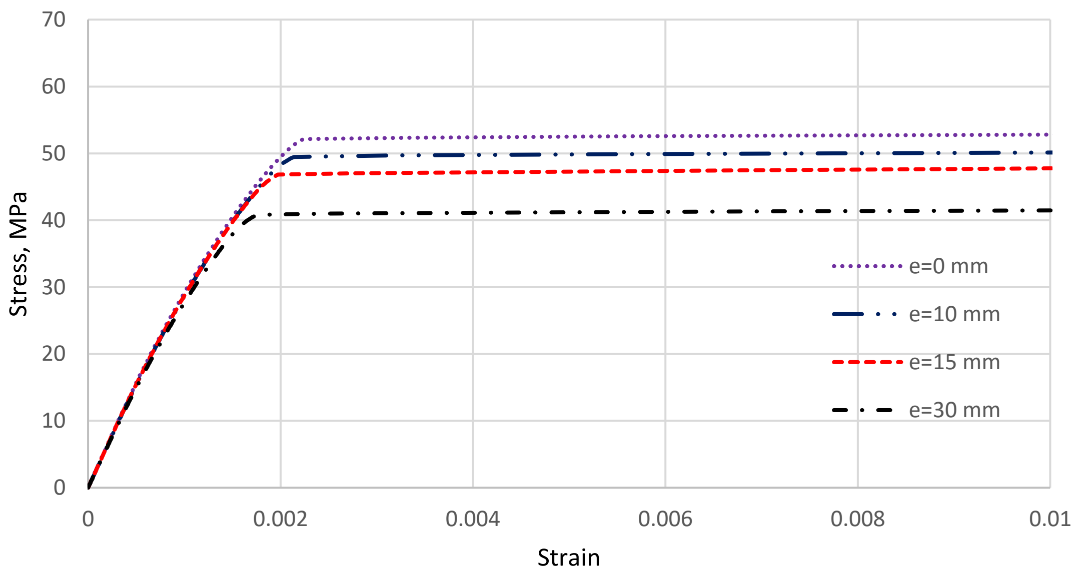


Figure 4-24 Effect of eccentricity of the load on compressive strength of a FRC column at 200℃

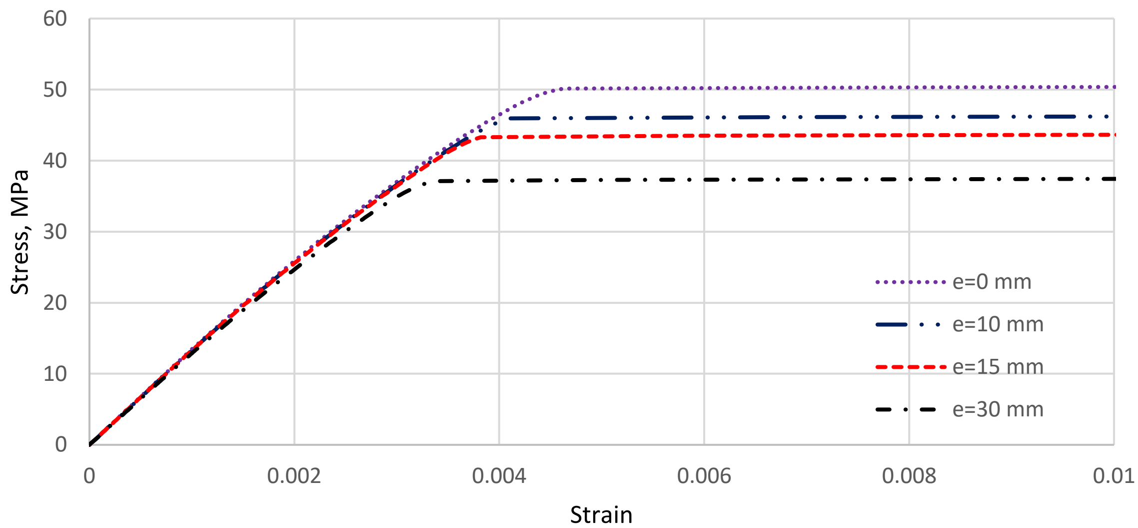


Figure 4-25 Effect of eccentricity of the load on compressive strength of a FRC column at 400℃

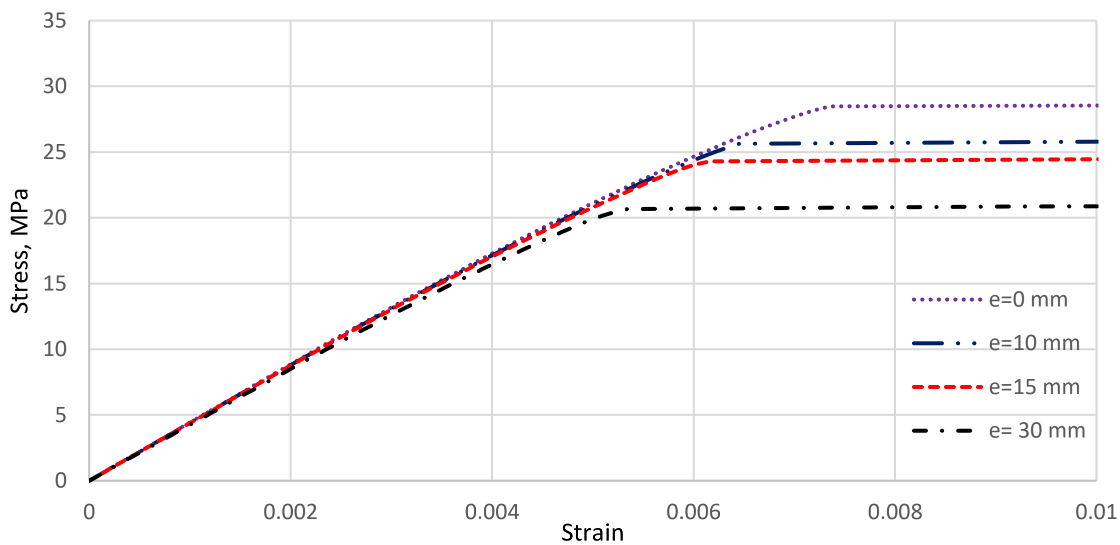


Figure 4-26 Effect of eccentricity of the load on compressive strength of a FRC column at 600℃

Table 4-8 Compression strength of FRC column at different eccentricities

Temp. ℃	20		200		400		600	
eccentricity	Compressive strength		Compressive strength		Compressive strength		Compressive strength	
	MPa	% Δ	MPa	% Δ	MPa	% Δ	MPa	% Δ
e = 0	56.3	0.0	52.8	0.0	50.4	0.0	28.5	0.0
e = 10	53.7	-4.7	50.1	-5.0	46.2	-8.3	25.8	-9.3
e = 15	51.0	-9.4	47.7	-9.6	43.7	-13.2	24.5	-13.9
e = 30	44.2	-21.6	41.5	-21.4	37.5	-25.6	20.8	-26.8

From Figures 4-23 to 4-26 and Table 4-8, it can be observed that generally the compression load carrying capacity of the column decreases with the increase of eccentricity. Flexural moment is created due to the eccentricity which reduces the compressive strength and generally the column is under combined action of axial force and flexural moment. The effect of eccentricity is slightly more pronounced at temperatures greater than 400℃ comparing to 200℃ and lower.

2.3.1.2 Sensitivity study at elevated temperatures

A sensitivity study at elevated temperatures is very essential in order to investigate the behavior of the FRC column at higher temperatures. The selected temperatures are 20℃, 200℃, 400℃ and 600℃. The effect of elevated temperatures on axial load carrying capacity of the column was studied by keeping eccentricity the same. The stress- strain curve for compression are plotted in figures 4-27 to 4-30. The results have been also tabulated in Table 4-9.

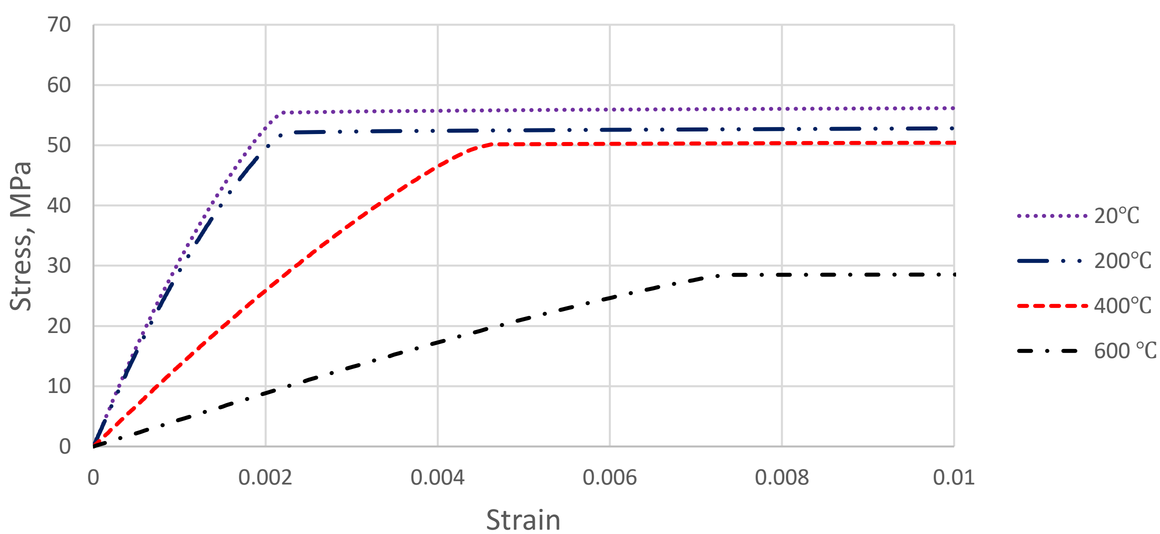


Figure 4-27 Effect of temperature on compressive strength of a FRC column at e=0 mm eccentricity

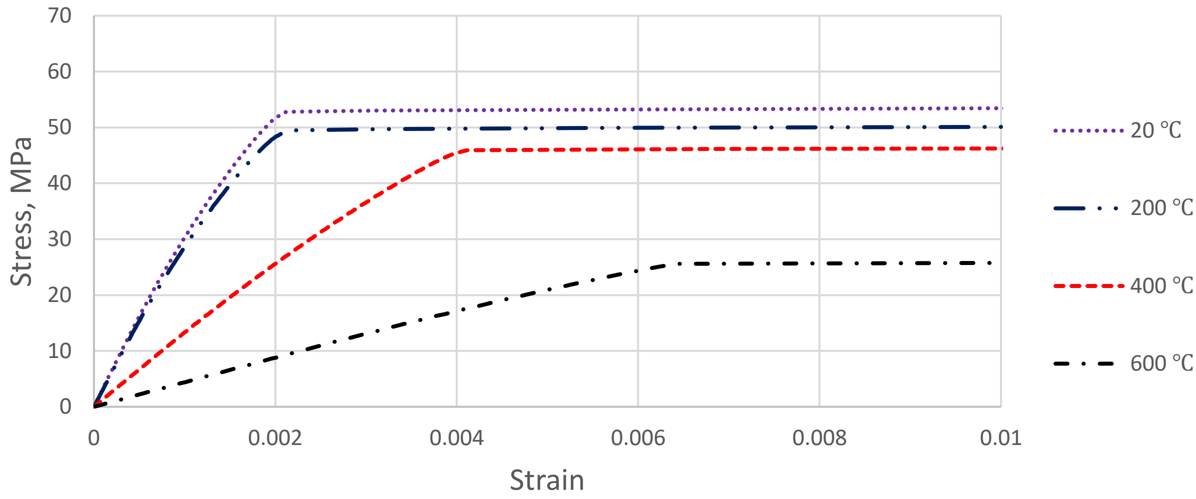


Figure 4-28 Effect of temperature on compressive strength of a FRC column at e=10 mm eccentricity

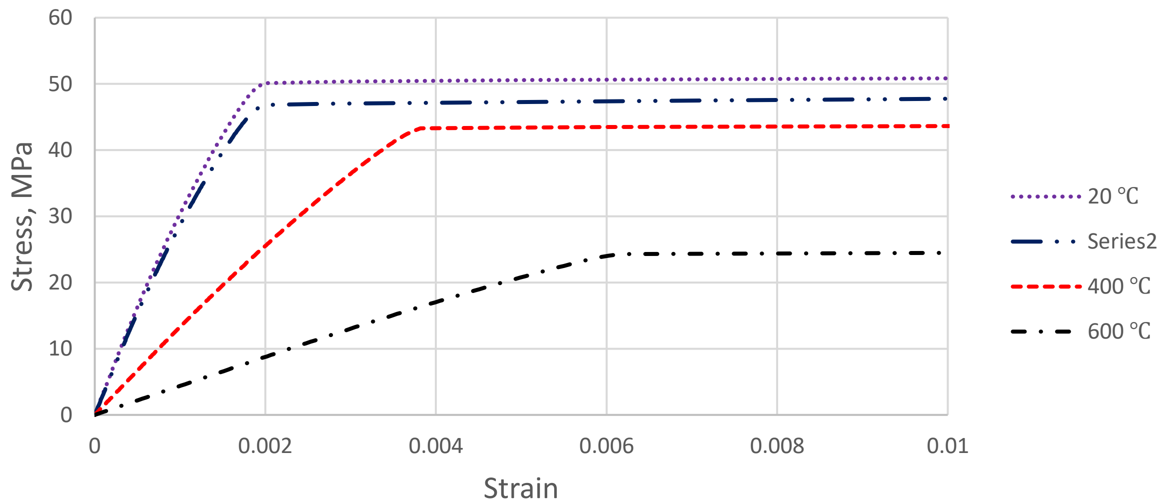


Figure 4-29 Effect of temperature on compressive strength of a FRC column at e=15 mm eccentricity

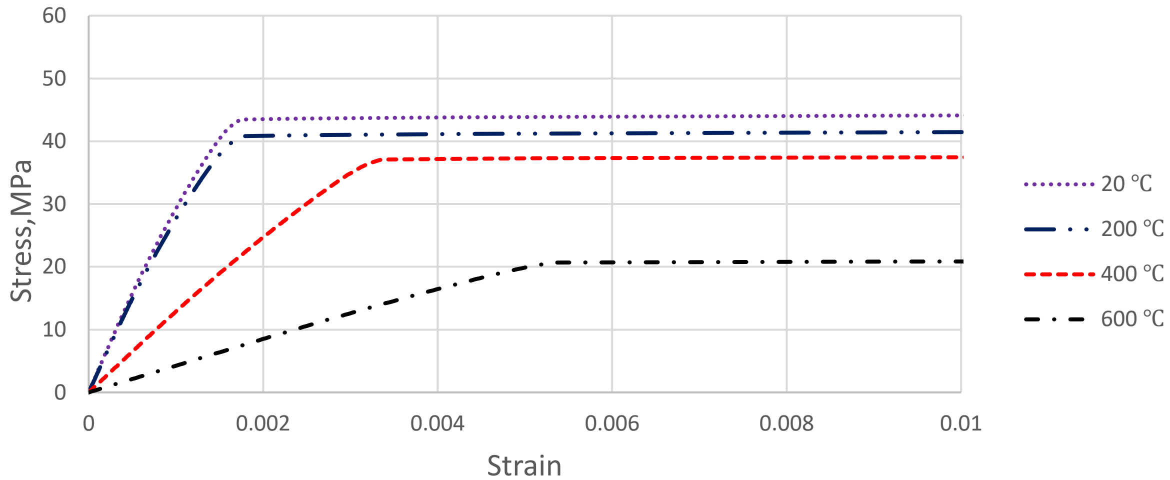


Figure 4-30 Effect of temperature on compressive strength of a FRC column at e=15 mm eccentricity

Table 4-9 Compression strength of FRC column at different temperatures

Eccentricity	e = 0		e = 10		e = 15		e = 30	
Temp. °C	Compressive strength		Compressive strength		Compressive strength		Compressive strength	
	MPa	% Δ	MPa	% Δ	MPa	% Δ	MPa	% Δ
20	56.31	0.00	53.66	0.00	51.01	0.00	44.16	0.00
200	52.78	-6.27	50.13	-6.58	47.70	-6.49	41.50	-6.02
400	50.35	-10.58	46.15	-14.00	43.72	-14.29	37.45	-15.19
600	28.48	-49.42	25.83	-51.86	24.51	-51.95	20.84	-52.81

With the increase of temperature the initial stiffness of the column is reduced and the compressive load carrying capacity of the FRC column has decreased by about 6% at 200℃, approximately by 14% at 400℃ and by around 50% at 600℃. The effect of temperature is almost constant for the different values of load eccentricity.

4.5.3 Failure modes

By observing the failure mechanisms of all the conditions of the column, two failure modes were identified. When the axial load is applied without any eccentricity at any temperature, the FRC column fails in compression by crushing. However for the loads applied with the eccentricity of 10 mm, 15 mm, and 30 mm, the failure is a combination of flexural and compression. Because of the end conditions of the column, i.e. pinned at the top and fixed at the bottom, the sensitive area of the column is close to the upper end with more bending and cracking noticed over there. The pattern of the cracks for the columns with eccentricities has shown that there is a development of a diagonal shear cracks. Due to the eccentricity of the load a flexural moment will be produced and in turn this moment will create a shear force on the column. Thus the column is subjected mainly to compression and bending but also to the shear force. As noticed on the properties of a cube and cuboid the tensile properties of the FRC decreases significantly with temperature, and these will result in the formation of shear cracks.

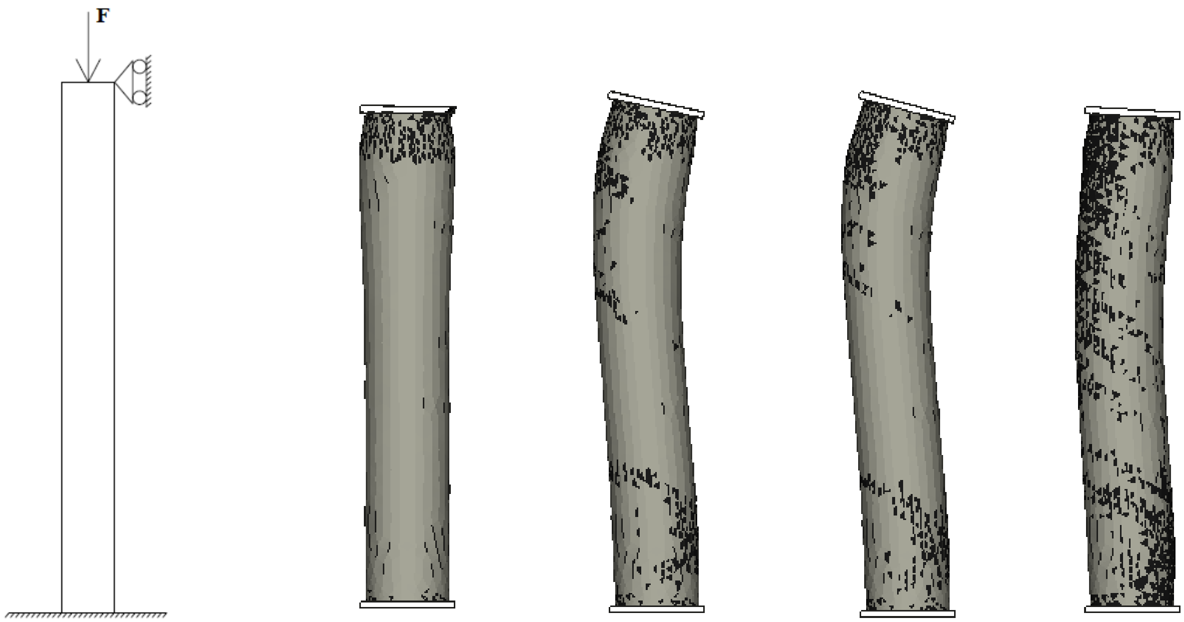


Figure 4-31 Failure modes of FRC column at 0 ℃ with eccentricities of 0 mm, 10 mm, 15 mm and 30 mm respectively.

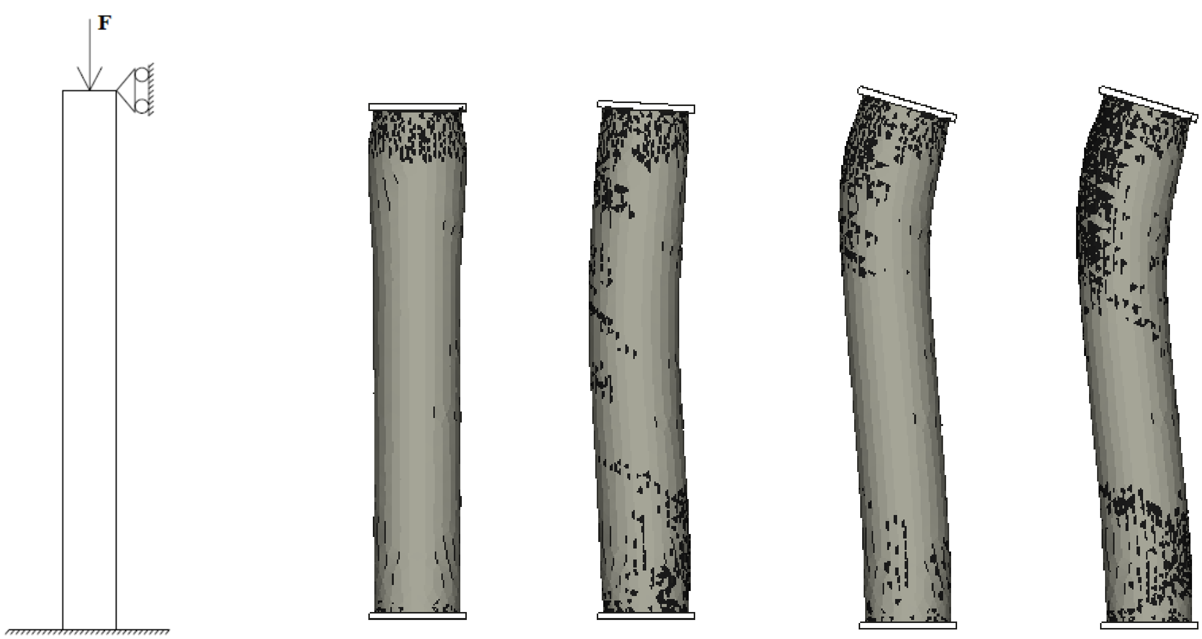


Figure 4-32 Failure modes of FRC column at 200 ℃ with eccentricities of 0 mm, 10 mm, 15 mm and 30 mm respectively.

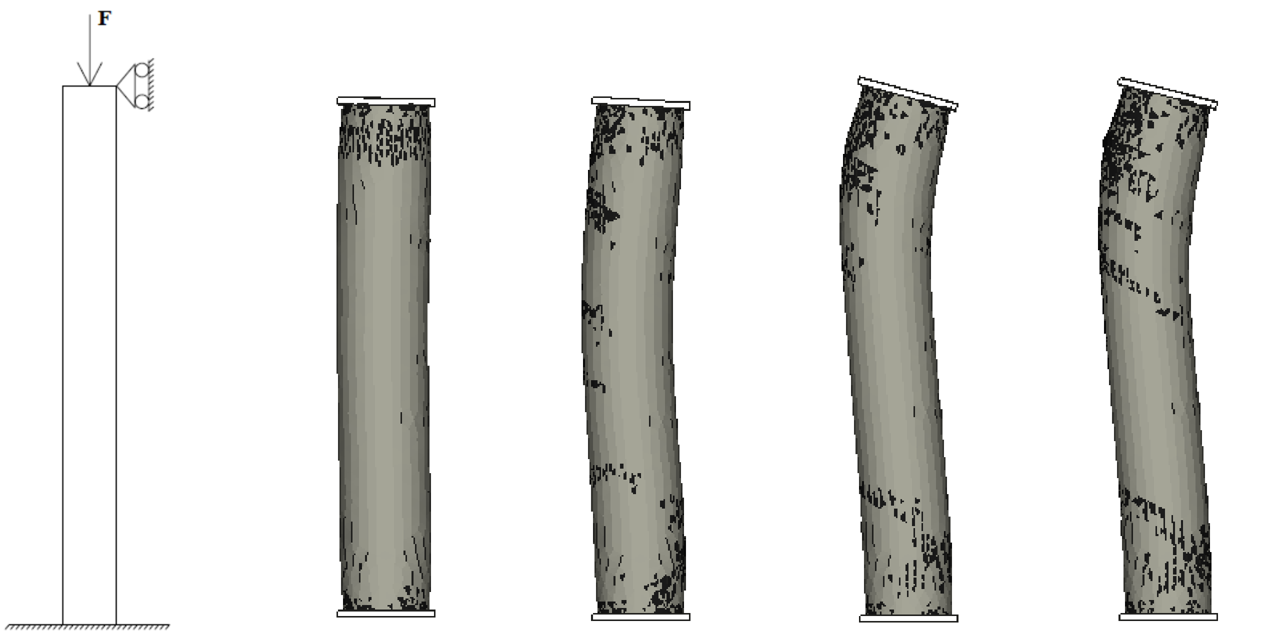


Figure 4-33 Failure modes of FRC column at 400 ℃ with eccentricities of 0 mm, 10 mm, 15 mm and 30 mm respectively.

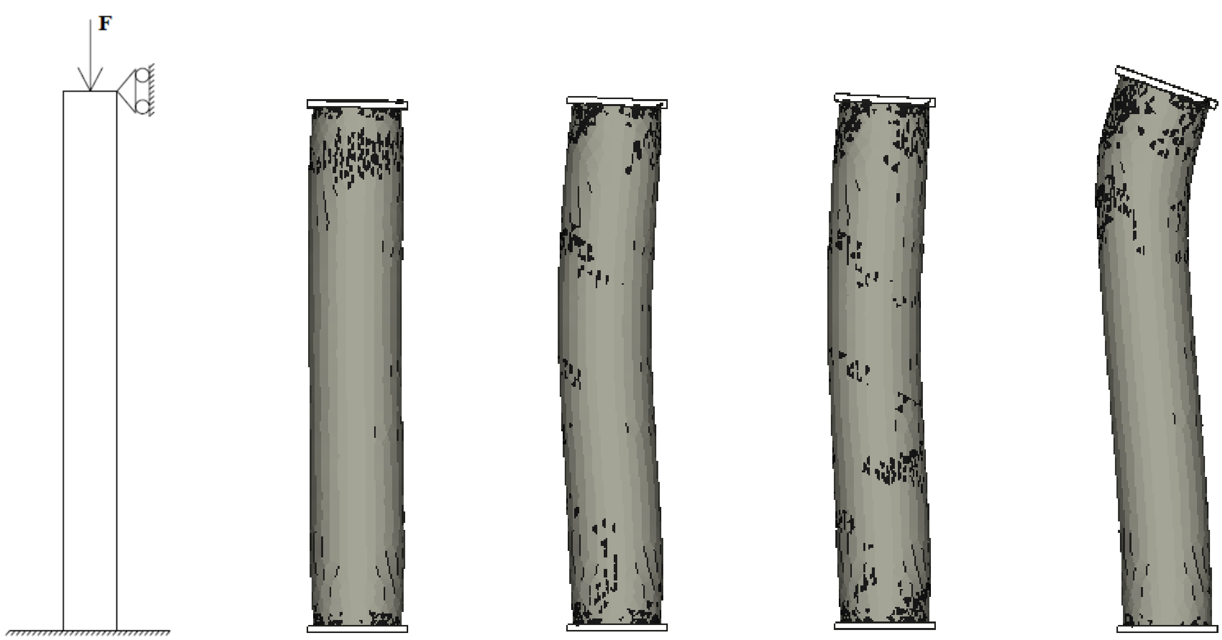


Figure 4-34 Failure modes of FRC column at 600 ℃ with eccentricities of 0 mm, 10 mm, 15 mm and 30 mm respectively.

5 CONCLUSIONS AND FUTURE WORKS

5.1 Conclusion and summary

In this work, the shear resistance of fiber reinforced concrete filled steel tubular column at elevated temperature was investigated. Only the concrete column was considered for simplification as steel and concrete separate from each other at elevated temperatures and most of the load is carried out by the concrete.

The pure shear strength of fiber reinforced concrete at elevated temperature was studied through numerical modelling. The accuracy of the numerical model was verified by comparing its predictions with the experimental results from Václav Ráček et al test. Satisfactory and realistic results were obtained in the shear strength, post crack behavior and similar failure of the fiber reinforced concrete by forming a diagonal crack. The validated model was employed for performing a parametric analysis, and based on the results the shear strength and post crack behavior of fiber reinforced concrete at elevated temperatures was identified.

Moreover, the tensile and compressive behavior of fiber reinforced concrete including the effects of characteristic size of the material was investigated using numerical modelling. Also a comparative study was done with the help of the model in order to compare the fiber reinforced concrete with the normal concrete.

Finally, knowing the shear strength, tensile strength and compressive strength of FRC, a full column was modelled numerically with the aim to investigate the failure behavior of a FRC column with an axial load applied with different eccentricities.

Considering the obtained results on this dissertation, it can also be concluded that:

- The ultimate shear strength of FRC is less than the ultimate tensile capacity of FRC. It is less than by about 33%, 39% and 53% for temperatures of 200℃, 400℃ and 600℃ respectively.
- The steel fibers improves the post crack behavior of FRC significantly.
- The compressive strength of the cube is about 20% higher than the compressive strength of the cuboid made from the same material. Unlike the compressive strength, the tensile strength of the cube and cuboid are almost the same.
- FRC column fails in compression by crushing when the axial load is applied without any eccentricity at any temperature.
- FRC column experiences a combined failure under the application of the load with some eccentricities.
- The orientation of cracks in the FRC column models with eccentric loading are aligned in a diagonal manner which indicates that the presence of a shear stress along with the flexural and compressive stresses.
- The failure modes of a column is greatly affected by the boundary condition and loading environment.

5.2 Future works

Along the development of this dissertation, it was noted a necessity for more information in some fields of application where some further investigations could be taken, such as:

- Experiments should be done at elevated temperature in order to fully understand the shear behavior of fiber reinforced concrete and can be used for the validation of the numerical models at the elevated temperature.
- Further experiments and investigations are necessary in order to study the influence of the percentage composition of steel fibers and their distribution on the concrete mix on the shear resistance of the concrete.
- At elevated temperature, in CFST columns steel and concrete separate because of the higher expansion of steel and most of the load is carried by the load. In this thesis a simplification is made by only considering the concrete but in reality there is a contact of steel and concrete at the time of failure. Therefore an advanced numerical model should be developed in order to investigate the full composite action of the steel and concrete for CFST columns at elevated temperature.
- To do a parametric analysis in order to investigate the influence of the load-level on the shear resistance of CFST column at elevated temperature.
- Another aspect which is important to study in the future is the effect of the boundary conditions of the column on the failure mode and the shear resistance of the column.

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