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Shear Capacity of Reinforced Concrete Beams at Elevated Temperatures

Mohamed Diab, *The University of Western Ontario*

Supervisor: Youssef, Maged, *The University of Western Ontario*

A thesis submitted in partial fulfillment of the requirements for the Master of Engineering Science degree in Civil and Environmental Engineering

SHEAR CAPACITY OF REINFORCED CONCRETE BEAMS AT ELEVATED TEMPERATURES

(Thesis format: Integrated Article)

by

Mohamed Diab

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Western University
London, Ontario, Canada

Abstract

Fire safety is an important design aspect that ensures structural integrity of a Reinforced Concrete (RC) building during fire events. As new codes are moving from prescriptive methods to performance-based methods, design engineers are in need of rational design tools to assess the capacity of RC elements during a fire event. Previous research work focused on the flexural and axial capacities of concrete elements exposed to fire. Research addressing the shear capacity of concrete elements exposed to fire is limited in the literature.

An analytical method to predict the shear capacity of RC beams exposed to elevated temperatures is proposed. The method assumes that shear capacity can be derived using existing ambient temperature methods while accounting for the effect of elevated temperatures on material properties. It involves heat transfer analysis, evaluation of the material properties at elevated temperatures, and application of the Modified Compression Field Theory (MCFT) to estimate the shear capacity. A parametric study is then conducted to investigate the effects of different parameters on the shear capacity of RC beams exposed to fire.

A simplified practical tool that can be used by design engineers to predict the shear capacity of RC beams exposed to elevated temperatures is proposed. Simple equations to calculate the average temperatures of the shear reinforcement and the concrete cross-section are the base for this tool. A set of design charts to determine the average temperature of shear reinforcement and concrete are also provided.

Keywords

Reinforced concrete, elevated temperatures, heat transfer, fire resistance, shear capacity, reinforced concrete beams.

Co-Authorship Statement

This thesis has been prepared in accordance with the regulations for an Integrated-Article format thesis stipulated by the School of Graduate and Postdoctoral Studies at the Western University. All analytical work presented in this thesis was performed by Mohamed Diab under supervision of Dr. M. A. Youssef. Drafts of all chapters were written by Mohamed Diab and modifications were done under supervision of Dr. M. A. Youssef. Chapters 3 and 4 of this thesis will be submitted to scholarly journals as manuscripts co-authored by Mohamed Diab and Maged Youssef.

Chapter 3: Analytical Prediction of the Shear Capacity of Reinforced Concrete Beams at Elevated Temperatures

A paper co-authored by Mohamed Diab and Maged Youssef will be submitted to Magazine of Concrete Research.

Chapter 4: Simplified Practical Method to Predict the Shear Capacity of Reinforced Concrete Beams Exposed to Fire

A paper co-authored by Mohamed Diab and Maged Youssef will be submitted to Engineering Structures.

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Last but not least, I cannot find suitable words to express my gratitude for my parents for their love, support, and sacrifices. I am indebted to them for everything I have. Without their encouragement and support, this achievement could not have been possible.

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List of Abbreviations and Symbols

b	cross-section width
c	concrete cover
C_c	specific heat capacity of siliceous concrete
E_s	initial modulus of elasticity of steel at ambient temperature
E_T	initial modulus of elasticity of steel at elevated temperatures
f'_c	compressive strength for concrete at ambient temperature
f_y	yield strength of steel bars at ambient temperature
f'_{cT}	reduced compressive strength at elevated temperatures
f_t	tensile strength for concrete at ambient temperature
f_{tT}	reduced tensile strength at elevated temperatures
f_{yT}	reduced yield strength of reinforcing bars at elevated temperatures
h	cross-section height
K_c	thermal conductivity of siliceous concrete
n_w	ratio between the surface temperature and the fire temperature
n_x, n_y	ratios between the internal and surface temperatures due to heating in the x

and y directions, respectively

t fire duration

t^* Equivalent fire duration assuming ISO 834 standard fire

T temperature in degree Celsius

T_{xy} temperature rise at any point located at (x, y)

T_f fire temperature

$T_{f\ (\mathrm{ISO})}$ ISO 834 standard fire temperature at a modified fire duration t^*

v_a aggregate interlock

v_c concrete contribution to the shear strength

v_{cz} compressive force within the concrete

v_d dowel action of the longitudinal reinforcing steel

v_s shear reinforcement contribution to the shear strength

v_n nominal shear strength

x, y horizontal and vertical coordinates for any point within the beam section, origin located at bottom left of the section

x_v, y_v boundaries of fire affected regions

ε_{tot} total concrete strain at elevated temperatures

ε_{tr} transient creep strain in concrete

ε_{oT} strain at maximum stress at elevated temperatures

ρ_l longitudinal reinforcement ratio

ρ_t transverse reinforcement ratio

Δt time step

$\Delta\xi$ triangular mesh width

Γ compartment time factor

Chapter 1

1 Introduction

This chapter provides a general background about fire safety and highlights the motivation of this study. Research objectives are then listed and the outline of the thesis is presented.

1.1 Background

Reinforced concrete is considered the most widely used construction material in the world. It has many advantages including: strength, durability, speed of construction, and cost-effectiveness. Concrete is a non-combustible material and has a low thermal conductivity, which adds fire resistance to its advantages (Denoel, 2007). When the outer perimeter of a concrete element is exposed to elevated temperatures, the cooler inner core maintains its load-bearing capacity. Also, the low thermal conductivity protects the reinforcing steel bars from the elevated temperatures. Regardless of these benefits, concrete resistance to fire should be accounted for in the design stages as its strength properties degrade when exposed to elevated temperatures.

A fire event can be a destructive force that causes many deaths and results in millions of dollars worth of property loss each year (Buchanan, 2002). For this reason, design for fire resistance has become a necessity. It ensures that the structure has sufficient capacity to support its loads for a period of fire exposure that guarantees safety during the evacuation and fire extinguishing processes (Lataille, 2003).

To improve fire design provisions, research work that investigates the behavior of reinforced concrete structures exposed to fire has received attention over the last four decades (Youssef and Moftah, 2007). This behavior was examined by fire tests and through the use of advanced analytical methods (Khoury, 2007). Fire tests are expensive and not feasible in early design stages. Advanced analytical methods are often based on using the Finite Element Method (FEM) (Lie, 1992). Although the FEM provides accurate modelling, it requires thorough knowledge about heat transfer analysis and the use of sophisticated computer programs. Thus, using the FEM may not be feasible for practicing design engineers.

Currently, concrete structures are being designed for fire safety using prescriptive methods that are based on experimental investigations. These methods usually specify the minimum cross-section dimensions and clear cover to achieve specific fire ratings. They tend to be conservative, and do not give engineers sufficient design flexibility. Moreover, they are not easy to apply to irregular shapes and innovative architectural solutions (Purkiss, 2007). As new codes are moving towards performance-based design, efficient design tools are needed to facilitate the prediction of the load-carrying capacity of reinforced concrete elements during exposure to elevated temperatures.

1.2 Motivation

Although a large number of experimental and analytical research programs were conducted on the fire resistance of reinforced concrete, the majority of this research work focused on their flexural performance. The literature is lacking experimental and analytical work addressing the shear capacity of reinforced concrete elements during fire conditions (Bamonte et al., 2009).

There is an increasing awareness that shear failure can be critical and may be the governing failure mode of reinforced concrete elements during fire. This has been proved by actual past incidents (Taewre, 2007). Shear failure is catastrophic as it occurs suddenly without sufficient warning. Therefore, it is essential to adequately evaluate the shear resistance of reinforced concrete elements during fire conditions to avoid such disastrous mode of failure.

This thesis addresses the lack of rational design tools to predict the shear capacity of reinforced concrete elements during fire. It builds on previous research work conducted at Western University that addressed the development of simplified tools to estimate the flexural and axial behavior of reinforced concrete members during fire conditions.

1.3 Research Objectives

Engineers are in need of guidelines and simplified tools to design reinforced concrete structures to ensure their structural integrity during fire events. There is a lack of tools that address the shear capacity of reinforced concrete elements during exposure to fire. This thesis aims to address this gap by developing a rational method that can evaluate the shear capacity of reinforced concrete beams during fire conditions. The specific objectives of the research are:

- To develop and validate an analytical method that can reasonably predict the shear capacity of reinforced concrete beams during exposure to a fire event;
- To conduct a parametric study to investigate the effects of various design parameters on the shear capacity of reinforced concrete beams during exposure to fire;
- To develop simplified equations to calculate the average temperatures that influence the shear capacity and use them to evaluate shear capacity.

1.4 Outline of the thesis

Chapter 2 presents background about fire safety principles and the current approaches to evaluate the fire resistance of reinforced concrete members. The chapter also reviews the shear resistance mechanism at ambient temperatures and discusses challenges in modelling the shear behavior at elevated temperatures. The importance of developing rational tools to predict the shear capacity of reinforced concrete elements during exposure to fire is highlighted.

Chapter 3 proposes a method to predict the shear capacity of reinforced concrete beams at elevated temperatures. This method utilizes the finite difference method (FDM) to predict the temperature distribution within a concrete cross-section, and uses the modified compression field theory (MCFT) to predict the shear capacity. The proposed methodology is validated using available experimental results found in the literature. The chapter also presents a parametric study that evaluates the effect of the different design parameters on the shear capacity at elevated temperatures. These parameters include amount of transverse and longitudinal reinforcement, cover, concrete strength, section dimensions, and fire duration.

Chapter 4 presents simplified practical tool to predict the shear capacity of RC beams exposed to fire conditions. The method begins with the development of simplified equations to calculate average temperatures of the shear reinforcement and the concrete cross-section. These calculated temperatures are then used to assess the corresponding deteriorated strength properties and to predict the shear capacity using the simplified formulas of the Canadian Standards Association (CSA A23.3-04). The proposed tool is validated using the predictions of the analytical method presented in chapter (3) and experimental results by others.

Chapter 5 provides a brief and general discussion of the thesis and states the important conclusions and recommendations for future work.

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Chapter 2

2 Literature review

Reinforced concrete is one of the most commonly used construction materials. It has many advantages, such as strength, durability, speed of construction, cost-effectiveness, and flexibility to form irregular and complicated shapes. Concrete has also excellent characteristics in regards to fire resistance. It is a non-combustible material that does not propagate fire or produce toxic gases (Denoel, 2007). Moreover, it can withstand high temperatures for a relatively long time because of its low thermal conductivity. When the surface of a concrete element is exposed to elevated temperatures, the cooler inner core maintains its load-bearing capacity. In addition, concrete protects the reinforcing steel bars.

Regardless of the mentioned benefits, concrete's resistance to fire should not be taken for granted. Concrete undergoes complex reactions and changes when exposed to elevated temperatures that result in a decline of its strength properties. Therefore, it is necessary to design reinforced concrete structures to have adequate fire resistance that addresses the structure integrity during exposure to fire.

This chapter presents the necessary background information about fire resistance of reinforced concrete structures. The chapter begins with a description of development stages of fire and fire safety requirements. Effects of fire on reinforced concrete including modes of failures are then discussed. The chapter also illustrates the different assessment methods of fire resistance and discusses the shear resistance mechanism at ambient temperature. Finally, the chapter presents challenges in modelling shear behavior at elevated temperatures, and highlights the need for developing tools that evaluate shear capacity of reinforced concrete structures during fire.

2.1 The development of a fire

Three elements have to exist for a fire to start. These elements are: oxygen, combustible material, and a heat source (Denoel, 2007). They can be represented by the fire triangle shown in Figure 2.1. The four stages of fire development are: ignition and growth, flashover, fully-developed fire, and decay (Purkiss, 2007), and are shown in Figure 2.2.

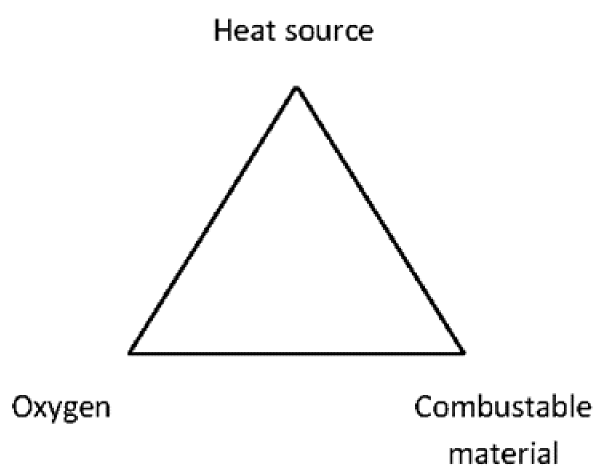


Figure 2.1 The fire triangle (Denoel, 2007).

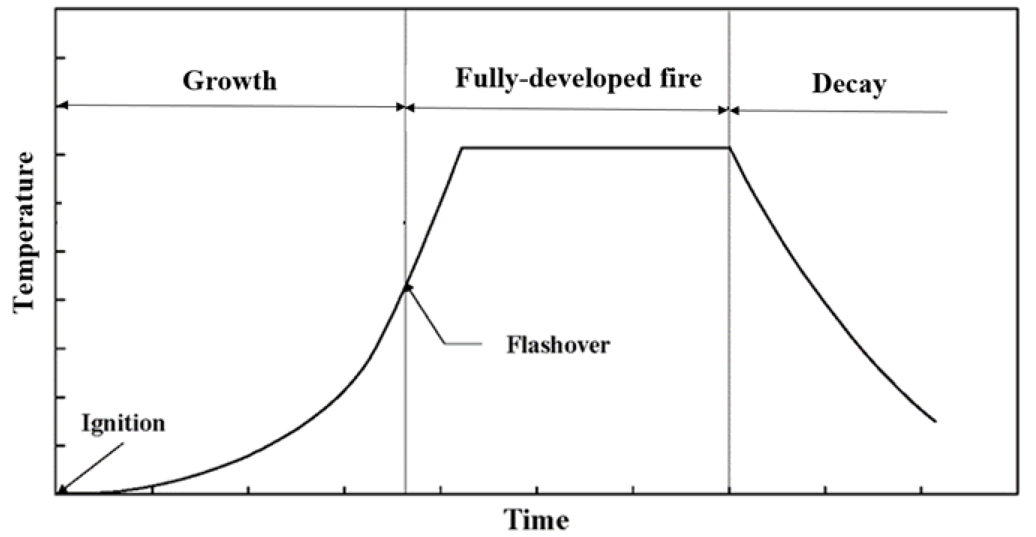


Figure 2.2 Fire development (Khoury, 2008).

2.1.1 Fire development stages

Fire originates when the three fire triangle elements exist in sufficient quantities to maintain combustion. Small amounts of the combustible materials start to burn producing gasses and smoke and allowing the fire to grow. A large number of fire incidents does not grow beyond this stage because of the lack of ventilation (oxygen source). The flashover phenomenon occurs when the air supply is significantly increased, which ignites all of the potential combustible materials and increases the temperature rapidly (Drysdale, 1998).

The fire then becomes fully-developed and produces a large amount of combustible gasses. During the fully-developed stage, structural elements are exposed to elevated temperatures, which may cause failures. The decay stage is characterized by a reduced combustion rate because of the lack of unburned combustible materials and declining temperatures.

2.1.2 Standard time-temperature curve

One common representation of the flashover and fully developed fire stages is the standard fire curve that represents the post-flashover stage, as shown in Figure 2.3. The most common standard time-temperature curves are ASTM-E119 and ISO-834 (or BS-476). Figure 2.4 shows that the two standard fire curves are almost identical.

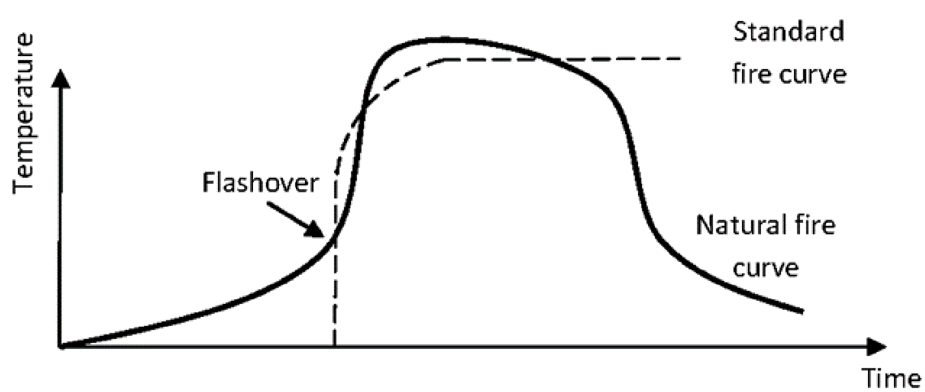


Figure 2.3 The standard fire curve as compared to a natural fire (Wit, 2011).

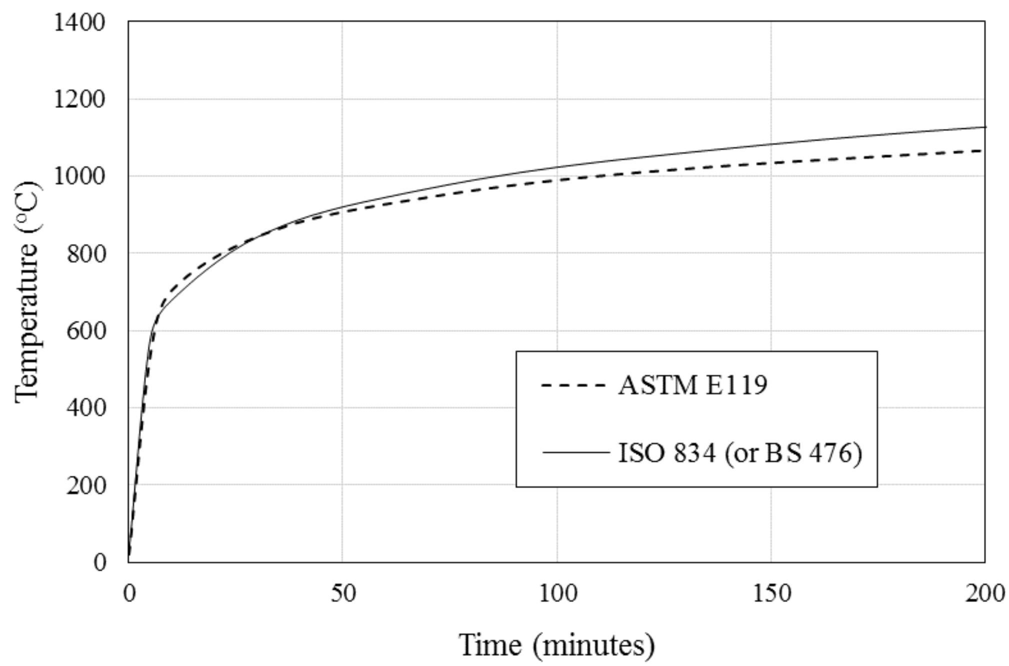


Figure 2.4 Standard fire curves.

2.2 Fire safety basics

Fires cause many deaths and millions of dollars' worth of property loss each year (Buchanan, 2002). In 2002, for instance, a total of 53,589 fire incidents, 304 fire deaths, 2,547 fire injuries, and a total of $1,489,012,263 in property losses were reported in Canada (CCFMFM, 2002). These numbers emphasize the necessity of implementing the fire safety requirements. Although there are several definitions for fire safety, it is widely defined as "*the application of scientific and engineering principles to the effects of fire in order to reduce the loss of life and damage to property by quantifying the risks and hazards involved and provide an optimal solution to the application of preventive and protective measures*" (Purkiss, 2007).

The key requirements for fire safety can be summarized as: (i) reducing the development and avoiding the spread of fire; (ii) ensuring safe and speedy evacuation of occupants; and (iii) maintaining a sufficient load-bearing capacity for structural elements for a specified period of time (Denoel, 2007).

Implementing preventive (active) and/or protective (passive) measures can achieve the fire safety requirements. Preventive, or active, measures include smoke detection systems, smoke alarms, and sprinklers. They assist in decreasing the spread of fire in the growth stage. Protective, or passive, measures include fire escape routes, compartmentation to contain the fire in the affected area, and structural fire safety. Design strategies often incorporate a combination of active and passive fire protection measures. Structural fire safety is concerned with the analysis, design, and construction of a building to ensure that the building can maintain its stability and has sufficient load bearing capacity for a reasonable duration of fire exposure (Lataille, J., 2003).

2.3 Effects of fire on reinforced concrete structures

Although concrete and steel are non-combustible materials, their strength and stiffness properties deteriorate due to exposure to elevated temperatures. Thermal expansion may also result in restraining forces. These effects lead to a reduction of the load-bearing capacity of the affected structural elements (Gabriel, 2000).

Modelling the behavior of concrete at elevated temperatures is a challenging task due to its non-homogenous composition (Fletcher, 2007). Concrete is a composite material made of a mix of aggregates, cement paste, and water. Each of these components has its own response to elevated temperatures leading to complex physical and chemical reactions of the whole mix (Gabriel, 2000).

Hertz (1992) explained the effects of elevated temperatures on concrete. Initially, when the temperature of concrete rises, the free water inside the concrete evaporates. This evaporation causes a pressure build-up within the concrete. When the temperature reaches about 150^{o}C, the water that is chemically bound to the hydrated calcium silicates is released. At a temperature of 300^{o}C, the aggregates expand and the cement paste starts to shrink. When the temperature reaches a value between 400^{o}C and 600^{o}C, the calcium hydroxide $Ca(OH)_2$ breaks down into calcium oxide CaO and water H_2O, creating additional water vapor and more pressure. This pressure may be too high and may result in cracking and spalling of the concrete.

The effects of elevated temperatures on steel reinforcing bars depend on the steel type and method of manufacturing (Dougill, 1983). These effects are mainly characterized by a loss in strength and stiffness. When steel is exposed to elevated temperatures, a transformation will occur in the crystalline structure of carbon-based steels, which leads to a reduction in the strength and stiffness of the steel material. At a temperature of 600^{o}C, the yield stress and modulus of elasticity of steel are reduced by about 60% (Fletcher, 2006).

2.3.1 Failure of reinforced concrete structures due to fire events

Fire events can result in severe consequences with respect to loss of life, injuries, and property damage. For instance, fire resulted in a partial collapse of an 8-story reinforced concrete building in Athens, Greece in 1980 (Franssen et al., 2004). The fire started at the 7th floor and rapidly spread within the building due to lack of compartmentation. Thermal expansion of different structural elements was considered the main cause for the failure. Fire also resulted in a partial collapse of a reinforced concrete 21-story office building in Sao Paulo, Brazil in 1987 (Franssen et al., 2004). Approximately two hours into the fire, the structural core of the building collapsed. Excessive thermal expansion of the beams caused the framing elements to fracture. These incidents illustrate the importance of structural fire safety to minimize the possibility of structural failure and catastrophic collapse during fire.

Local failures due to exposure to fire can be categorized into the following modes: bending, buckling, compression, anchorage (bond), shear, and spalling (Khoury, 2007). Horizontal structural elements such as beams and one-way slabs usually experience bending failure. On the other hand, buckling and compression failures are considered to be the most common mode of failure in vertical elements such as columns (Khoury, 2007). Anchorage failure is defined as when reinforcing bars are pulled out of the concrete due to insufficient embedment length and/or bond (Khoury, 2007). Spalling can be defined as the breaking of concrete layers from the element surface due to exposure to high temperatures (Kodur, 2008). The spalling phenomenon may lead to excessive exposure of reinforcement, resulting in a rapid strength reduction of the reinforcement. Moreover, spalling results in a reduction of the element cross-section. High-strength concrete is more vulnerable to spalling due to its low permeability compared to normal strength concrete (Khoury, 2000).

2.4 Methods of assessment of fire resistance

Over the last few decades, several research studies were conducted to assess the performance of reinforced concrete structures during fire. Depending on the level of complexity and accuracy, methods to assess the fire resistance of RC structures can be classified into experimental, prescriptive, analytical, and simplified (Khoury, 2007).

2.4.1 Experimental methods

Standard fire tests involve evaluating the structural performance of an element using a furnace. The fire temperature is controlled to follow a specific standard time-temperature curve. The element is considered fire resistant if it satisfies specific load capacity and/or deformation criteria (Lataille, 2003). The major disadvantages of standard fire tests are the cost and needed time (Bailey, 2002). However, standard fire tests are the only valid method to assess deficiencies in construction detailing and to assess fire performance of new materials (Gabriel, 2000). Results of fire tests are also essential to validate various numerical tools (Moftah, 2008). Full-scale fire tests can trace the global response and behavior of a structure while emphasizing on special features, such as effects of restraints and continuity.

Lin et al. (1981) tested eleven RC beams using ASTM E119 (1995) standard fire to study the influence of beam continuity, moment redistribution, and aggregate type. Continuous beams were found to have higher fire resistance than simply supported beams because of moment redistribution.

Lin and Ellingwood (1987) tested six RC beams to study the flexural behavior of RC beams when exposed to ASTM E119 standard fire. The authors found that the temperature history in reinforcement steel bars is the most critical factor influencing the overall strength of the beams.

Shi et al. (2004) conducted an experimental program on six RC beams with different concrete cover thickness. The study concluded that increasing the bottom concrete cover improves the fire resistance of reinforced concrete flexural members as it protects the steels bars from the fire temperature. Cracking of the concrete in the tensile zone reduces the effectiveness of the concrete cover.

2.4.2 Prescriptive methods

Prescriptive methods provide guidelines and tabulated data to specify minimum cross-section size and concrete cover to maintain the resistance of an element for a predefined fire exposure duration (Moftah, 2008). They are mainly developed based on standard fire tests. Their main advantage is their ease of application (Fletcher, 2007). On the other hand, they tend to be conservative, and do not give engineers the flexibility to use irregular shapes and innovative solutions (Purkiss, 2007). Lie (1992) discussed the prescriptive tools used by different building codes.

2.4.3 Analytical methods

Advanced calculation methods imply conducting a complete thermal and mechanical analysis to evaluate the resistance of a structural element exposed to fire. The thermal analysis is conducted using the principles of the heat transfer, which involves considering the temperature-dependent thermal properties of concrete (Youssef et al., 2009). Mechanical analysis is conducted based on the structural mechanics principles, using the deteriorated strength properties of materials due to exposure to elevated temperatures (Purkiss, 2007).

The Finite Element Method (FEM) is considered the most common tool to carry out advanced fire resistance analysis (Rigberth, 2002). Advanced calculation methods account for the continuous alteration of the thermal and mechanical characteristics of the materials, the boundary conditions, and the non-homogenous distribution of the temperatures within the elements. Although advanced calculation methods provide a very realistic modelling, they require thorough background knowledge and the use of sophisticated computer programs.

Kodur and Dwaikat (2009) presented a macroscopic finite element model for tracing the performance of RC structural members during fire. Their model was validated by comparing its predictions with the results of full-scale experimental tests. The study suggested that a fire scenario has a noticeable effect on the fire resistance of structural elements. Kodur and Wang (2004) conducted a similar study to predict the fire resistance of high strength concrete columns. The predictions of this study showed a good agreement with the results of full-scale experimental tests.

Terro (1998) developed finite element computer programs to predict the behavior of three-dimensional reinforced concrete structures in fire. The programs were validated by comparing their predictions with the results of experimental testing. Lie (1992) described the utilization of numerical tools to determine the temperature distribution within cross-sections of structural elements and outlined how these numerical tools are used to calculate fire resistance. The detailed thermal and mechanical properties of concrete and steel were also detailed.

2.4.4 Simplified methods

Simplified methods use the ambient-temperature design methods while taking into account the effects of fire (Fransen et al., 1995). They should provide structural engineers with reliable performance-based fire design tools.

Malhotra (1972) presented a performance design method for fire resistance by using empirical temperatures distribution charts. Wade (1991) presented tools that can evaluate the performance of reinforced concrete elements exposed to fire. These tools were based on the empirical temperature distribution charts and the degraded strength properties of the concrete and steel bars at elevated temperatures.

El-Fitiany and Youssef (2009) developed a sectional analysis method to predict the flexural and axial behavior of RC members exposed to fire conditions. The method was validated using tests results found in the literature. Based on the results of an extensive parametric study, El-Fitiany and Youssef (2011) proposed equivalent stress-block parameters for elements exposed to elevated temperatures. The sectional analysis method was then expanded to assess the behavior of continuous RC beams when exposed to fire (El-Fitiany and Youssef, 2014). El-Fitiany (2013) presented simplified methods to construct the interaction diagrams of fire-exposed elements and to analyze RC frames exposed to fire.

2.5 Shear strength of RC beams at ambient temperatures

The flexural behavior of reinforced concrete has been extensively investigated and many well-defined design tools have been developed. However, there is a lack of agreed upon theory regarding shear behavior. Empirical equations to estimate shear resistance are still the basis of many codes of practice (Bentz and Collins, 2006).

2.5.1 Shear resistance mechanism

Prior to flexural cracking, uncracked concrete carries all the shear forces. After developing flexural cracking, the shear is resisted by the following components (Regan, 1993): (i) shear in the compressive zone, V_{cz}; (ii) shear resisted by the dowel action of the longitudinal reinforcing steel, V_d; (iii) interface shear transfer along two faces of the cracks (aggregate interlock), V_a. After the development of inclined cracks, the shear reinforcement resists part of the shear forces, V_s. Upon the yielding of the stirrups, the inclined cracks widen rapidly, which decreases the contribution of the aggregate interlock. Eventually, the concrete element experiences failure either by splitting or by the failure of the compression zone of concrete. The internal forces developed in a typical concrete beam at a crack location are shown in Figure 2.5.

The shear failure of the concrete beams is brittle and occurs without sufficient advance warning. Thus, the main purpose of shear reinforcement is to ensure that the flexural capacity of the concrete element is achievable. Prior to developing inclined cracking, there is no significant contribution of the stirrups. However, after forming the inclined cracks, the stirrups play a noticeable role in resisting the applied shear forces (Winter and Nilson, 1979). The role of stirrups in enhancing the shear resistance is achieved by providing an additional shear force. Shear reinforcement can also maintain the influence of the aggregate interlock by controlling the widths of the inclined cracks. Generally, the nominal shear strength of concrete, Vn, is written in the following form:

$$V_n = V_c + V_s$$

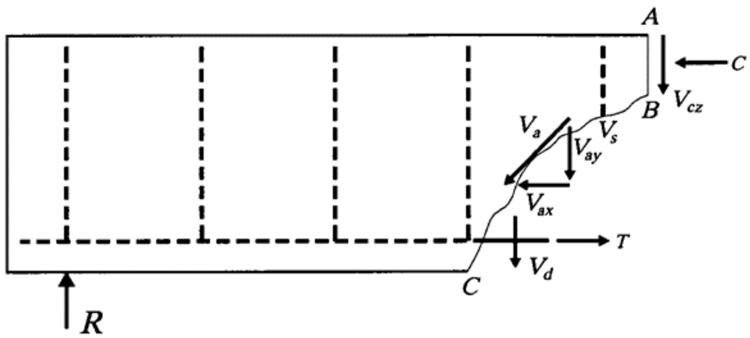


Figure 2.5 Shear resistance mechanism.

2.5.2 Shear models for reinforced concrete beams

Several research attempts have been conducted to develop a rational modelling of the shear behavior of reinforced concrete beams. The ASCE-ACI Committee 445 (1998) has provided a summary of the developed shear models. These models can be classified into two main groups:

- Models based on an equilibrium approach:
    - The 45^{o} Truss Model
    - The Variable-Angle Truss Model

    - The Modified Truss Model
- Models based on a compression field approach:
    - The Compression Field Theory
    - The Modified Compression Field Theory
    - The Rotating-Angle Softened Truss Model
    - The Fixed-Angle Softened Truss Model

One of the earliest models developed to analyze the shear behavior of reinforced concrete beams was the “45-degree truss model” developed by Ritter (1899) and Morsch (1902). The truss model depicts a beam as a truss in which the compression chord represents the concrete compression zone, the tension chord represents the longitudinal steel, the vertical truss members represent the stirrups, and the diagonal truss members represent the concrete compression struts developed between the inclined cracks. The inclination angle of these diagonals was assumed to be a 45^{o} angle with respect to the horizontal beam axis. This model was modified by Lampert and Thurlimann (1968), who proposed the variable-angle truss model.

In 1974, Mitchell and Collins presented the diagonal compression field theory for members subjected to torsion, which introduced the softening effect of concrete. Afterwards, Vecchio and Collins (1981) presented the compression field theory for members subjected to shear. This theory considers the equilibrium, compatibility, and stress-strain relationships of concrete and steel to predict the shear behavior of concrete elements. The theory assumes that the inclination of the principal stresses coincides with the inclination of the principal strains. This theory was further developed by Vecchio and Collins (1986) to present the modified compression field theory, which accounts for the contribution of the tensile strength of concrete.

2.5.3 The modified compression field theory

The MCFT was developed based on experimental observations of a large number RC members loaded in shear. Hence, the method has been widely recognized by the international community and forms the basis of shear provisions in the Canadian standards and the American Association of State Highway and Transportation Officials (AASHTO).

The MCFT is capable of accurately capturing the complex response of RC members subjected to shear. It uses equilibrium, compatibility, and stress-strain relationships to predict the relationships between shear and normal stresses and the resulting deformations. The theory also takes into consideration the tensile stresses in the concrete between the cracks. Figure 2.6 provides a summary of the MCFT equations (Bentz and Collins, 2006). The MCFT assumes that the direction of the principal stresses coincides with the direction of the principal strains and that the critical cracks are also inclined in that direction. The MCFT has been successfully implemented in a number of computer programs such as Response 2000 (Bentz, 2000).

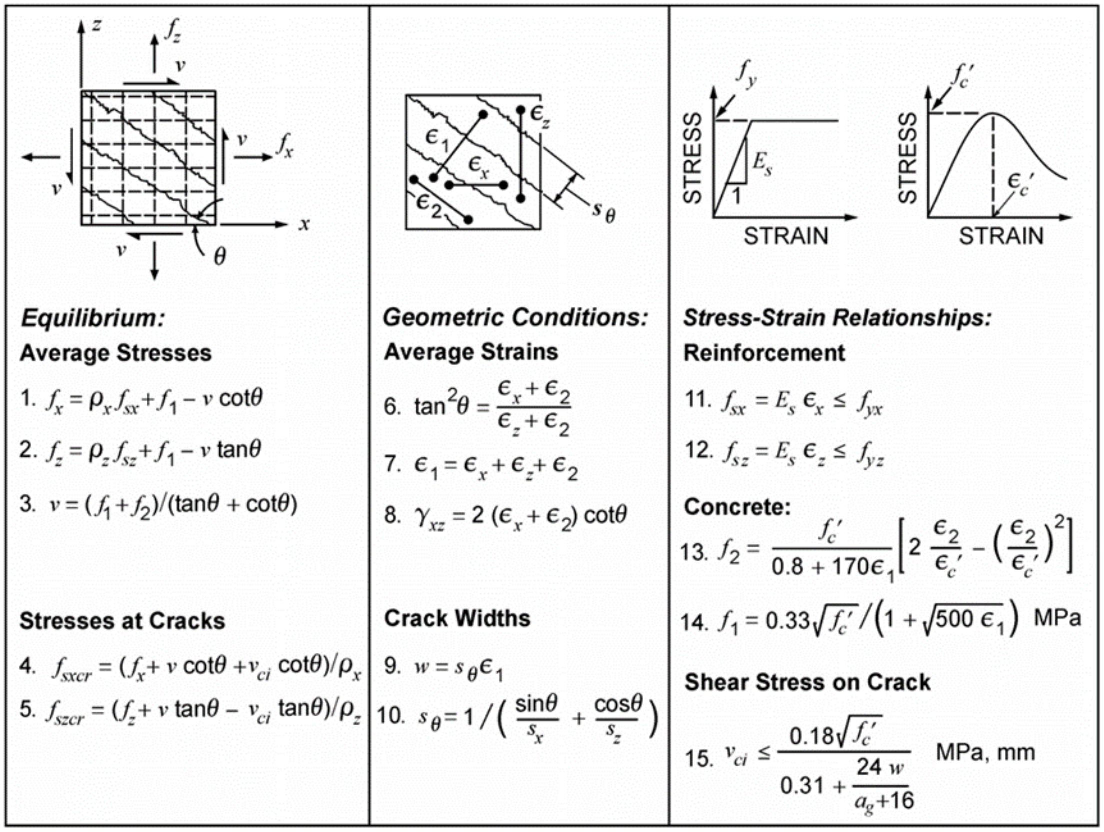
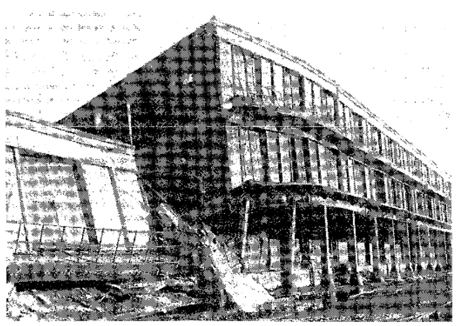


Figure 2.6 A summary of the relationships used in the modified compression field theory (Bentz and Collins, 2006).

2.6 Shear capacity at elevated temperatures

Actual past fire events have proved the severity of shear failure due to fire (Taewre, 2007). Figure 2.7 shows examples of shear failures during fire exposure. A 3-story reinforced concrete building in the port of Ghent, Belgium collapsed in 1974 after about an hour and half of fire exposure (Taerwe, 2007). Thermal expansion of beams and slabs resulted in excessive drifts of the building columns causing them to fail in shear. Another example of shear-caused failure occurred in 1996 in the city library of

Linkoping, Sweden (Anderberg, 1996). A similar failure was observed in the U.S. military personnel records center building (Beitel, 2002).



Port of Ghent warehouse (Taerwe, 2007)

Library of Linkoping (Anderberg, 1996)

US military personal records center building (Beitel, 2002)

Figure 2.7 Examples of shear failures due to fire exposure.

2.6.1 Complexity of shear behavior assessment at elevated temperatures

Modelling the complex interaction of the shear resistance mechanisms at elevated temperatures is exceptionally difficult. Fire increases the height of cracks, and thus reduces the depth of the compressive zone (Xiang et al., 2012). The reduced strength of the reinforcing steel decreases the value of its resisted shear. Moreover, the reduced bond strength between concrete and steel due to exposure to fire increases the height and width of cracks. This reduces the effectiveness of the dowel action of tension steel and the influence of the aggregate interlock.

2.6.2 Previous research addressing shear capacity at elevated temperatures

Most of the previous research studies focused on the flexural performance of structural elements exposed to elevated temperatures. Although, there is an increasing awareness that shear failure can be critical and may be the governing failure mode of reinforced concrete elements during fire (Bamonte et al., 2009), shear resistance has not been thoroughly investigated. There is lack of availability of well validated computational methods to assess the shear behavior of concrete elements subjected to fire (Bamonte et al., 2009).

El-Hawary et al. (1997) conducted experimental research on eight RC beams, of 200 x 120 mm cross-section size, to study the effect of fire duration and concrete cover on their shear behavior. The beams were divided into two groups: (i) beams with a cover thickness of 20 mm, and (ii) beams with a cover thickness of 40 mm. The beams were exposed to different durations of fire, then cooled by water. Afterwards, the beams were loaded till failure. It was found that the compressive strength of concrete decreased as the fire exposure time increased. It was also found that increasing the concrete cover helped to decrease the fire damage on the shear capacity of the beams. Saafi (2002) proposed a design method to predict the residual flexural and shear capacities of FRP reinforced concrete beams exposed to fire. The method involved determining the distribution of the elevated temperatures within a cross-section and then using the degraded strength properties of FRP and concrete due to elevated temperatures. It was found that the FRP temperatures decrease with increasing the concrete cover, and FRP reinforced concrete beams exhibited significant degradation in their strength during exposure to fire. Shang et al. (2009), as reported by Xiang (2012), carried out experimental research to investigate the effect of fire on the shear resistance of eight simple beams strengthened with U-type high performance ferrocement laminate layers. Liu et al. (2009) proposed an analytical tool to predict the flexural and shear capacities of RC beams strengthened by a carbon fiber sheet exposed to fire and validated the method by test results found in the literature.

Desai (1998) tested five RC beams that were exposed to fire to evaluate their shear capacity. All beams were 200 x 300 mm in cross-section and had 1600 mm overall length, and 1400 mm supported span. The beams were exposed to fire from three sides and loaded until they failed in shear. The study found that the shear capacity during fire is affected by the reduction of the strength properties of the concrete and the shear reinforcement. Masaad et al. (2007) proposed a calculation method for the shear capacity of prestressed hollow core concrete slabs exposed to fire. This method relies on the assumption that shear capacity at elevated temperatures can be evaluated using the ambient temperature methods while considering the reduced strength properties of materials.

2.6.3 Research needs

The provisions of the current design codes of practice are set out to provide recommendations for the minimum sizes of structural elements and the cover to the reinforcement steel. In most of the cases, these guidelines are based on ensuring the flexural capacity of a structural member (Holly et al., 2011).

As the cross-section size is one of the main factors that contribute to the shear capacity of the structural elements, the minimum sizes set by the codes of practice for fire resistance are expected to provide adequate shear capacity. However, this is not clearly mentioned in the design codes. Due to the complexity of evaluating shear capacity at elevated temperatures, there is a need to develop a reliable, simple, and cost-effective method to estimate the reduced shear capacity of reinforced beams exposed to elevated temperatures. This tool can also eliminate the need to perform full-scale laboratory fire testing which is very expensive.

2.7 Summary

Reinforced concrete is one of most commonly used construction materials due to its excellent properties including good fire resistance. However, concrete resistance to fire should not be taken for granted.

The basics of fire safety and the effects of fires on reinforced concrete were reviewed in this chapter. The different methods for evaluating the fire resistance of reinforced concrete structural elements and the key findings of previous research work on the fire response of reinforced concrete members were presented. It was observed that the research work focused mainly on the flexural performance of structural elements. Shear resistance, in contrast, was not thoroughly investigated. This chapter presented a summary of shear resistance mechanisms at ambient temperature. In addition, effects of elevated temperatures on shear resistance were discussed.

The current codes of practice and standards are based on using prescribed methods for fire safety design of reinforced concrete structures. As new codes are moving towards performance-based design, there is a critical need for further research leading to the development of rational design tools for the fire resistance. There is a clear lack of computational methods to assess the shear behavior of concrete elements exposed to fire. The main objective of this research, therefore, is to address this lack.

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Chapter 3

3 Analytical prediction of the shear capacity of reinforced concrete sections at elevated temperatures

This chapter proposes an analytical method to predict the shear capacity of Reinforced Concrete (RC) sections exposed to elevated temperatures. The limitations of current design codes and the importance of this chapter are first summarized. The proposed method is then explained. It incorporates heat transfer analysis, evaluation of the material properties at elevated temperatures, and application of the Modified Compression Field Theory (MCFT) to estimate the shear capacity. The method is validated using experimental results by others. The proposed method is then utilized to conduct a parametric study that evaluates the effects of different parameters on the shear capacity at elevated temperatures. The studied parameters are: transverse reinforcement and longitudinal reinforcement ratios, concrete cover and strength, beam height and width, and fire duration.

3.1 Introduction

Reinforced concrete is one of the most commonly used construction materials. It has advantages including strength, durability, speed of construction, and flexibility to be formed into irregular and complicated shapes. Concrete has also excellent characteristics with respect to fire resistance. It is non-combustible, and, thus does not propagate fire, nor produce toxic gases (Denoel, 2007). Its low thermal conductivity delays the increase of temperatures of the inner core of the cross-section and the reinforcing steel bars. Regardless of these benefits, concrete resistance to fire should not be taken for granted.

Concrete undergoes complex reactions and changes when exposed to elevated temperatures, resulting in decline of its strength properties.

Fire resistance of reinforced concrete structural elements can be determined either by conducting experimental tests or by performing analytical studies such as the Finite Element Method (FEM) (Kodur et al., 2004). The cost of experimental fire studies is very high, which makes them unsuitable for regular design. The FEM requires performing a coupled thermal-stress analysis, which cannot be easily performed by design engineers.

Currently, concrete structures are being designed for fire safety using prescriptive methods that are based on experimental investigations. These methods usually specify the minimum cross-section dimensions and clear cover to achieve specific fire ratings. They tend to be conservative, and do not give engineers any design flexibility (Purkiss, 2007). As new codes are moving towards performance-based design, simplified methods are needed to facilitate the prediction of the load-carrying capacity of reinforced concrete elements during exposure to elevated temperatures. El-Fitiany and Youssef (2009) developed and validated a sectional analysis method to predict the flexural and axial behavior of RC members exposed to fire conditions. El-Fitiany and Youssef (2014) utilized the sectional analysis method to analyze continuous RC beams exposed to fire.

Shear resistance during fire has not been thoroughly investigated. There is a lack of analytical methods that can assess the shear behavior of concrete elements exposed to fire (Bamonte et al., 2009). The provisions of the current design standards are set based on the flexural capacity of a structural member during fire (Holly et al., 2011). It is necessary that codes of practice implement methods of assessment for the shear capacity at elevated temperatures.

The aim of this chapter is to develop a reliable and simple method to estimate the reduced shear capacity of reinforced sections exposed to elevated temperatures. The method involves the use of heat transfer analysis to determine the temperatures within the cross-section, and the use of the MCFT (Vecchio and Collins, 1986) to predict the shear capacity. The chapter also presents a parametric study to investigate the effects of various design parameters on the shear capacity of reinforced concrete beams during exposure to fire.

3.2 Proposed method

The proposed method relies on the fact that shear capacity can be derived using existing ambient temperature methods while accounting for the effect of elevated temperatures on the materials (Masaad et al., 2007; and Katarina et al., 1994). Figure 3.1 summarizes the main steps for this method. Heat transfer analysis is performed to determine the temperature distribution within the beam cross-section. The reduced strength properties of concrete and steel bars are calculated. The shear resistance of the cross-section is calculated using the MCFT (Vecchio and Collins, 1986) for each time step of the fire duration. The method is further simplified by using the average temperatures for the steel bars and concrete.

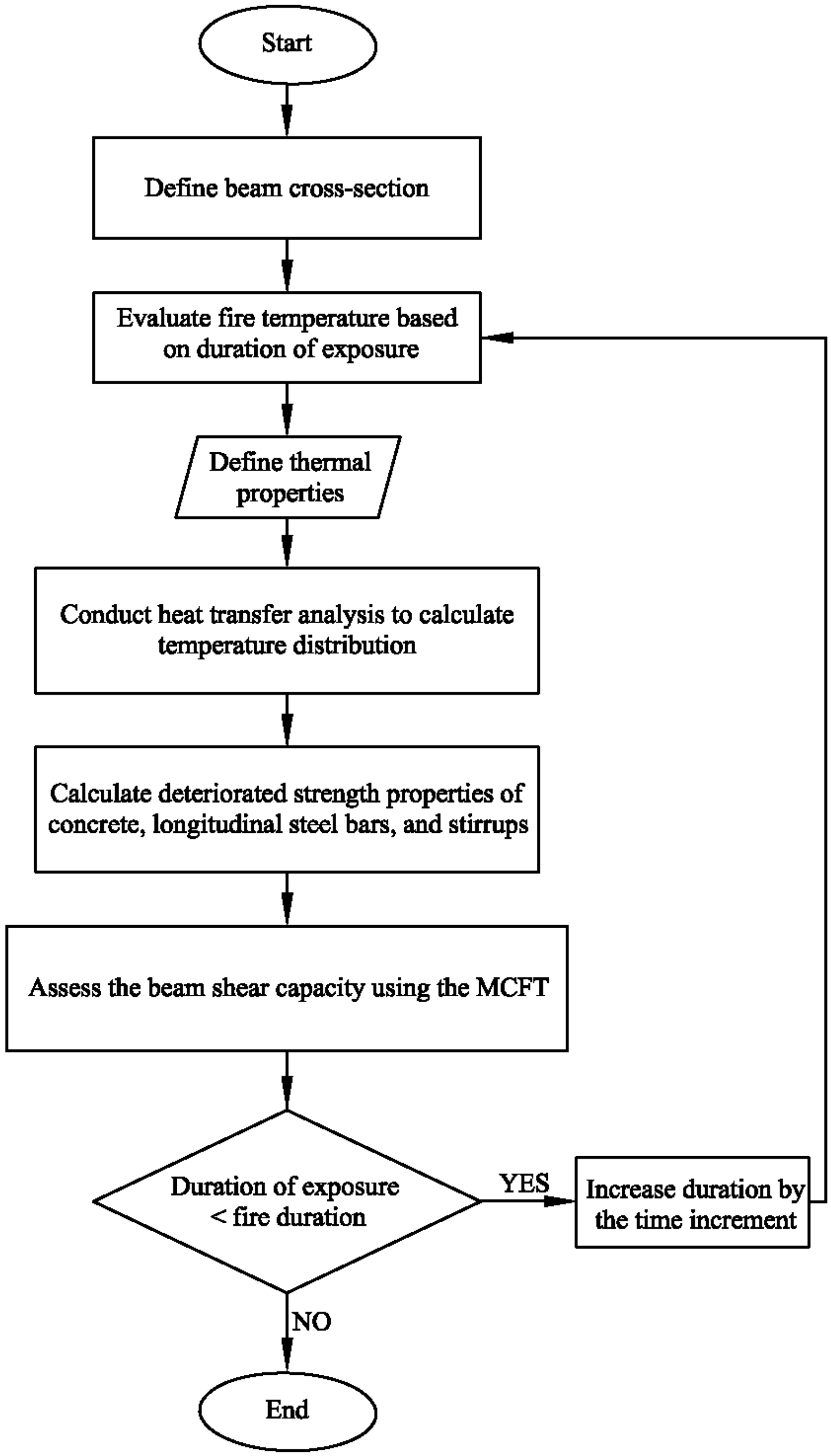


Figure 3.1 Prediction of shear capacity of a concrete beam exposed to fire.

3.2.1 Heat transfer analysis

There are several methods to predict the temperature distribution within a concrete cross-section during exposure to fire. Among those methods are the FEM (Lie, 1972), simplified methods (Wickstrom, 1986), and the FDM (Lie, 1992). The FEM is the most general tool to predict the thermal distribution but it is time consuming. The FDM is considered to be a simplified version of the FEM. A computer program using FDM, developed by El-Fitiany and Youssef (2009), is used to determine the temperature distribution within the RC beams cross-sections. The studied concrete section is divided into a number of 45^o mesh elements as shown in Figure 3.2. The temperature at the center of each element represents the temperature of the entire element. The thermal conductivity (k_c) and specific heat capacity (C_c) were assumed using the equations reported by Lie (1992).

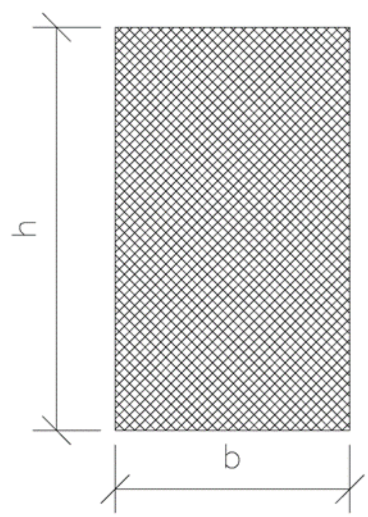


Figure 3.2 Typical heat transfer mesh.

The boundary conditions including dimensions, number of exposed faces, and the fire duration are first defined. The incremental temperature increase at the surface of the cross-section is determined based on the relationship between the fire temperature and its duration. Part of the heat energy conveyed to the boundary elements is used to increase their temperatures while the remaining energy is transferred to the inner elements. The effect of moisture content is included (Lie, 1992).

3.2.2 Properties of materials at elevated temperatures

Youssef and Moftah (2007) provided an assessment of the available models of the material properties of concrete and steel at elevated temperatures. Their recommended models are utilized in this research and are summarized below.

3.2.2.1 Concrete compressive strength

Hertz's model (Hertz, 2005) is used to predict the reduced concrete compressive strength at elevated temperatures (f'_{cT}):

$$f'_{cT} = \frac{f'_c}{1 + \frac{T}{15,000} + \left(\frac{T}{800}\right)^2 + \left(\frac{T}{570}\right)^8 + \left(\frac{T}{100,000}\right)^{64}} \tag{3.1}$$

Where (T) is the temperature in degrees Celsius, and (f'_c) is the concrete compressive strength at ambient temperature.

3.2.2.2 Concrete tensile strength

Youssef et al. (2008) recommend the use of Eq. 3.2 to predict the tensile strength of concrete at elevated temperature (f_{tT}).

$$f_{tT} = f_t \times \frac{f_{cT}^{'}}{f_c^{'}} \tag{3.2}$$

Where (f_t) is the concrete tensile strength at ambient temperature. The same equation is utilized in this research with (f_t) being defined using Eq. (3.3) (Bentz, 2000).

$$f_t = 0.45 \times (f_c^{'})^{0.4} \tag{3.2}$$

3.2.2.3 Fire-induced strains in concrete

Total concrete strain at elevated temperatures (ε_{tot}) is the summation of three terms: instantaneous stress related strain (ε_{oT}), thermal strain (ε_{th}), and transient creep strain (ε_{tr}). Thermal strain is the free thermal strain resulting from fire temperature and is affected by drying shrinkage. The study focuses on unrestrained elements, and, thus the thermal strain does not have an effect. Transient creep strain is induced during the heating of concrete and is considered the largest component of the total strain. It expresses the concrete capacity to relax external compressive stresses. Terro's model (Terro, 1998) is used to predict the transient creep strain at elevated temperatures.

$$\varepsilon_{tr} = \varepsilon_{03}\left(0.032 + 3.226\frac{f_c}{f_c^{'}}\right)\left(\frac{V_a}{0.65}\right) \tag{3.4}$$

Where, V_a is the volume of aggregate

$$\varepsilon_{o3} = 43.87\times10^{-6} - 2.73\times10^{-8}T - 6.35\times10^{-8}T^2 + 2.19\times10^{-10}T^3 - 2.77\times10^{-13}T^4$$

3.2.2.4 Yield stress of Steel

Lie's model (Lie, 1992) is used to predict the reduced yield strength of reinforcement steel at elevated temperatures (f_{yT}):

$$f_{yT} = \left[1 + \frac{T}{900\ln(T/1750)}\right] \times f_y \qquad 0 < T \le 600^o C \tag{3.5}$$

$$f_{yT} = \left[\frac{340 - 0.34T}{T - 240}\right] \times f_y \qquad T > 600^o C \tag{3.6}$$

3.2.2.5 Young's modulus of steel

Lie's model (Lie, 1992) is used to predict young's modulus of steel at the elevated temperature (E_T):

$$E_T = \left[1 + \frac{T}{2000 \times \ln\left(\frac{T}{1100}\right)}\right] \times E_s \qquad 0 < T \le 600^o C \tag{3.7}$$

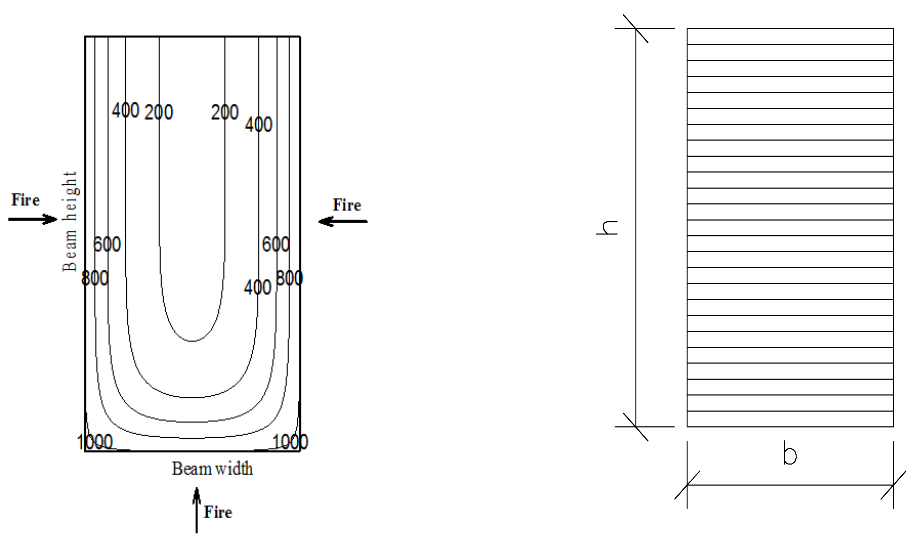
$$E_T = \left[\frac{690 - 0.69 \times T}{T - 53.5}\right] \times E_s \qquad T > 600^o C \tag{3.8}$$

3.2.3 Prediction of shear capacity

The Modified Compression Field Theory (MCFT) was developed by Vecchio and Collins (Vecchio and Collins, 1986; Collins and Mitchell, 1991) based on experimental observations of a large number of RC members. It uses equilibrium, compatibility, and stress-strain relationships, to predict the relationships between shear and normal stresses and the resulting deformations. The MCFT has been successfully implemented in a number of computer programs. One of these programs is Response 2000 (Bentz, 2000), which is used in this study.

3.2.4 Temperatures within concrete and steel

When a beam is exposed to fire from three sides, the pattern of temperature distribution within the beam cross-section takes the shape of the contour lines shown in Figure 3.3 (a). This varying temperature within the beam cross-section poses a challenge while defining the temperature to be used to calculate the reduced strength properties of materials. The following sub-sections propose a solution for this challenge.



a) Contour lines b) Layers

Figure 3.3 Temperature distribution for a beam heated from three sides.

To utilize the method of sectional analysis to define the flexural response of RC sections during fire exposure, El-Fitiany and Youssef (2009) divided the concrete section into layers and assigned an average temperature to each layer, Figure 3.3 (b). The same approach is valid while evaluating the shear behavior because the MCFT can be applied using a layered approach (Vecchio and Collins, 1988).

A further simplification that involves the use of one average temperature for the whole section is examined in this section. Two models were developed for a number of concrete sections, Figure 3.4. In the first model, each section is assumed to have five layers with different temperatures assigned to each layer. The shear capacity of the section is then evaluated using the MCFT and compared to the capacity evaluated based on one average temperature. Table 3.1 gives the details of the six analyzed sections. Figure 2.5 compares the shear capacities of the two models. The shear capacities predicted by the models are almost identical with maximum difference of 4%.

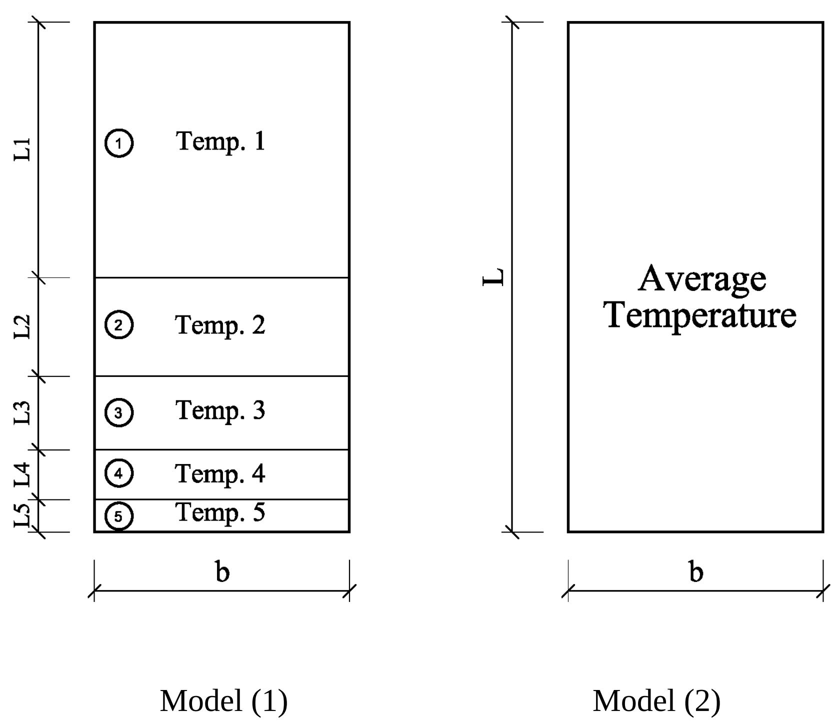


Figure 3.4 Average temperature in concrete.

Table 3.1 Details of two models for six concrete sections

Beam No.	Beam Dimensions & Stirrups (mm)	Model (1)				Model (2)	
		Layer No.	Layer height (mm)	Temp. (^oC)	Shear Capacity (kN)	Average Temp. (^oC)	Shear Capacity (kN)
1	200 x 300 φ6@200mm	1	150	358	87	420	85
		2	60	370			
		3	40	436			
		4	25	555			
		5	25	760			
2	200 x 300 no stirrups	1	150	358	48	420	50
		2	60	370			
		3	40	436			
		4	25	555			
		5	25	760			
3	200 x 400 φ6@200mm	1	200	355	118	404	121
		2	80	363			
		3	54	395			
		4	35	501			
		5	31	731			
4	200 x 400 no stirrups	1	200	355	65	404	63
		2	80	363			
		3	54	395			
		4	35	501			
		5	31	731			
5	200 x 500 φ9@200mm	1	250	350	194	391	190
		2	100	355			
		3	65	370			
		4	45	455			
		5	40	690			
6	200 x 500 no stirrups	1	250	350	84	391	81
		2	100	355			
		3	65	370			
		4	45	455			
		5	40	690			

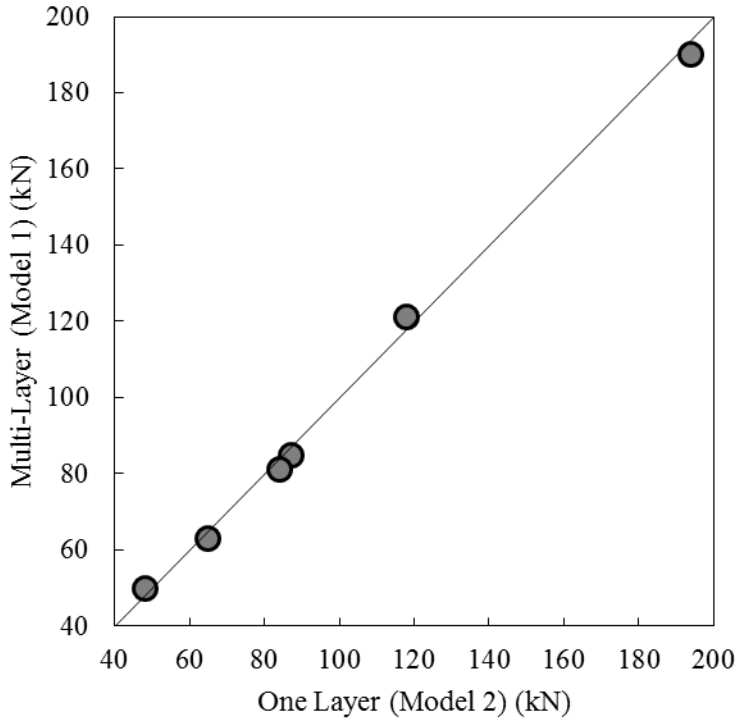


Figure 3.5 Comparison of shear capacities for different beams using multi-layers (Model-1) and one layer (Model-2).

As the vertical legs of the shear reinforcement are the main part providing shear resistance, their temperatures are used to calculate the material properties. Temperatures at mesh points that lie on the vertical legs were obtained from the heat transfer analysis. The average of these temperatures is then calculated. The use of such average is justified because the distance between the mesh points are equal and shear stresses are assumed to be constant along the stirrup height. The average temperature was then utilized to calculate the reduced strength properties of the shear reinforcement.

3.3 Validation of the proposed method

The proposed method to estimate the shear capacity of RC beams during exposure to fire is validated by comparing its predictions with the experimental and analytical data found in the literature. It has been quite difficult, however, to find literature discussing the shear resistance of reinforced concrete beams exposed to fire. The majority of the research on fire resistance of concrete structural elements has mainly been conducted to study flexural resistance rather than shear resistance.

3.3.1 Desai et al. (1998) experimental work

Desai et al. (1998) tested five RC beams that were exposed to fire from three sides to evaluate their shear capacity. All beams were 200 x 300 mm in cross-section and had 1600 mm overall length, and 1400 mm supported span. The beams were reinforced using 3φ20 mm and 2φ12 mm at their bottom and top sides, respectively. Concrete cover was 25 mm. Table 3.2 gives details about the stirrups and the concrete compressive strength for the tested beams. Figure 3.6 shows the cross-section of beam B501. Desai et al. (1998) proposed the use of a center bar to enhance the fire resistance as this bar is well protected by surrounding concrete. The tensile strength of the steel reinforcement (f_y) was 460 N/mm^2. The beams were exposed to ISO 840 fire curve while supporting 70, 80, 90, 110, and 120 kN at the middle of the span of beams B102, B202, B301, B401, and B501. All tested beams failed in shear. Analysis steps for beam B501 are given as an example of the application of the proposed method.

Table 3.2 Details of beam specimens (Desai et al., 1998)

Beam No.	f_c' (N/mm^2)	Center bar	Stirrups
B102	39.40	-	-
B202	34.40	1φ16 mm	-
B301	42.00	1φ20 mm	-
B401	42.00	-	φ6mm@200 mm
B501	38.50	1φ16 mm	φ6mm@200 mm

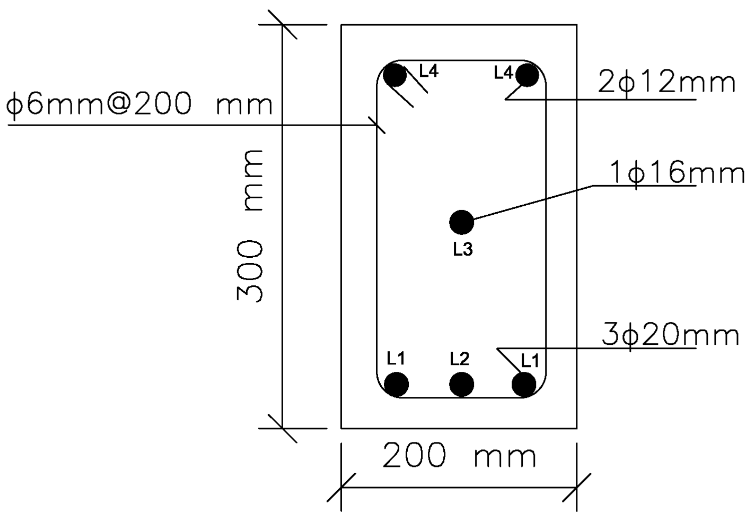


Figure 3.6 Cross-section of beam B501.

3.3.1.1 Temperatures in concrete and steel during exposure to fire

Heat transfer analysis was performed to determine the temperature distribution within the cross-section of the beam. The section was divided into 45^{o} mesh elements and the temperature for each element was obtained. The cross-section was then divided into 120 layers and the average temperature for each layer is calculated. Figure 3.7 shows the average temperatures for the whole section at different fire durations. The average temperature in concrete reaches 617^{o}C after approximately two hours of fire exposure.

Figure 3.8 shows the increase in temperatures in the steel bars during the fire event. Locations of steel bars are identified in Figure 3.6. The temperatures of bars located at L_1, L_2, L_3, and L_4 reached 643, 565, 370, and 460 ^{o}C, respectively, after two hours of fire exposure. It can be seen that bars L_1 and L_2 experienced a higher temperature increase, than other steel bars, as they are located close to the fire-exposed faces of the beam. On the other hand, L_3 (the center bar) experienced the lowest temperature increase.

Temperatures resulting from heat transfer analysis were recorded along the vertical leg of the stirrups at 4 mm spacing. The average temperature within the leg were then calculated. Figure 3.8 shows that the average temperature in stirrups reaches 640^{o}C after two hours of fire exposure.

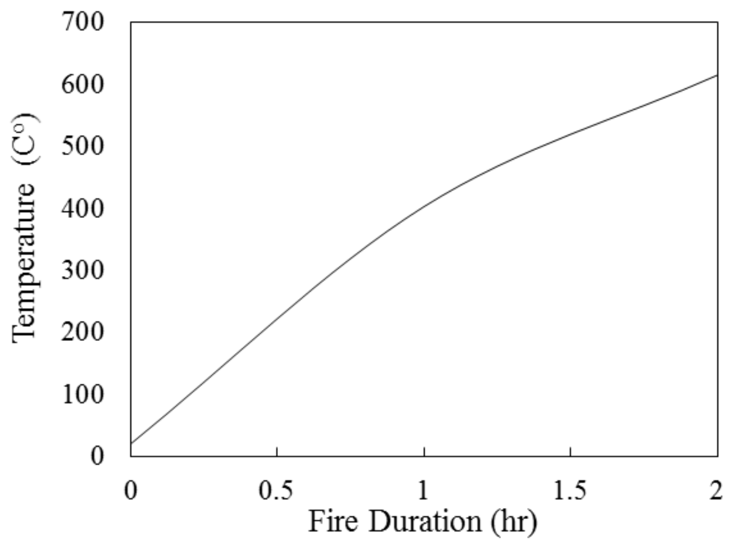


Figure 3.7 Average temperature in concrete during fire exposure.

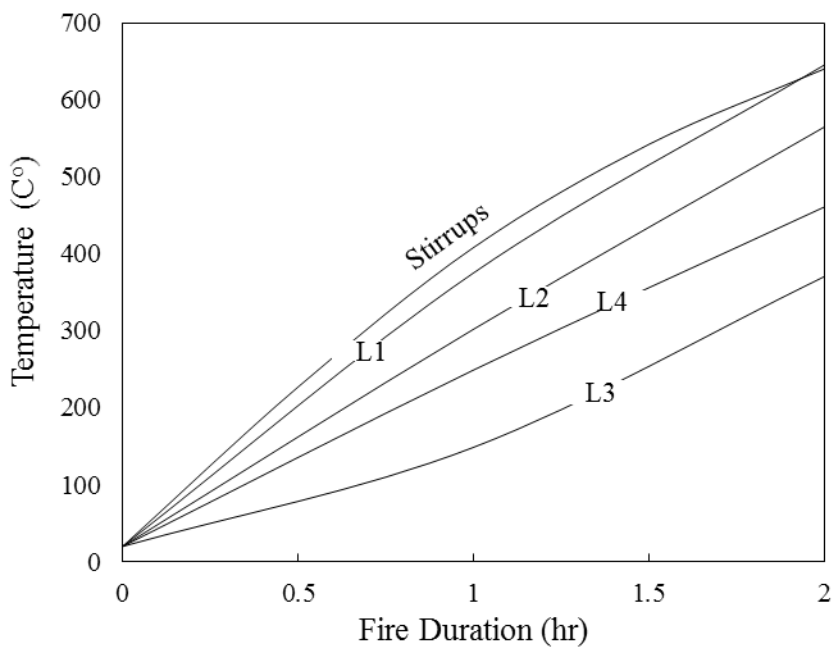


Figure 3.8 Temperature in steel bars during fire exposure.

3.3.1.2 Properties of materials during fire

The strength properties for steel and concrete during fire exposure are shown in Figure 3.9 to Figure 3.12. The stirrups and the tension steel, located at the bottom side of the beam, experienced high reduction in strength properties as shown in Figure 3.9 and Figure 3.10. Figure 3.9 shows that the yield stress of the stirrups decreased by 70% after two hours of fire exposure as compared to only 26% decrease for the center bar. Figure 3.10 shows that the Young's modulus of the stirrups decreased by 59% after two hours of fire exposure as compared to a decrease of only 17% for the center bar. Figure 3.11 shows that the concrete compressive strength decreased by 69% after two hours of fire exposure.

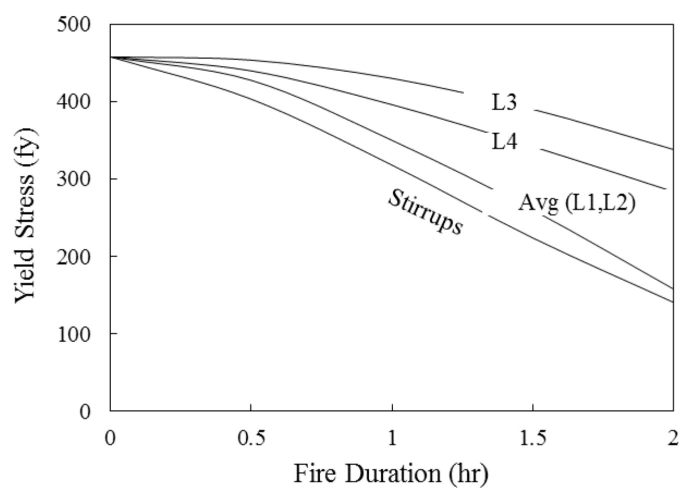


Figure 3.9 Reduction in yield stress of steel bars during fire exposure.

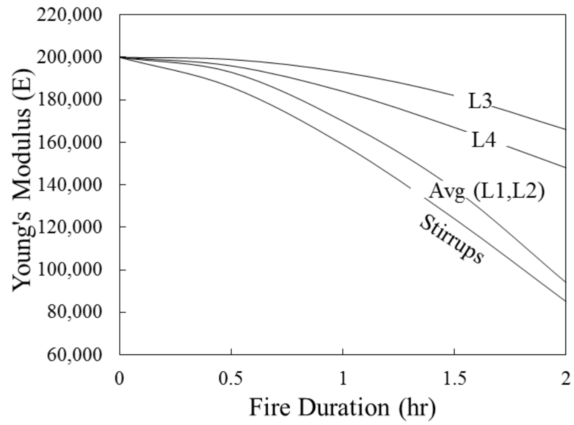


Figure 3.10 Reduction in Young’s modulus of steel bars during fire exposure.

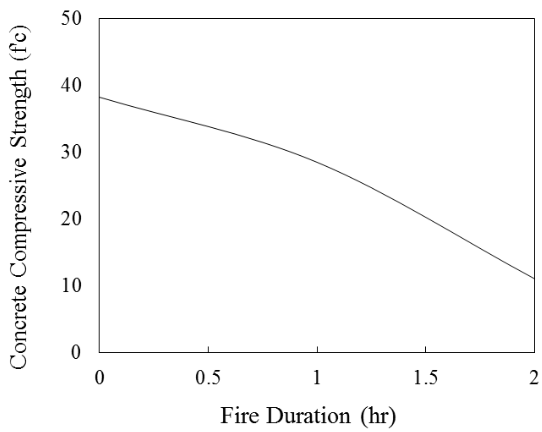


Figure 3.11 Reduction in concrete compressive strength during fire exposure.

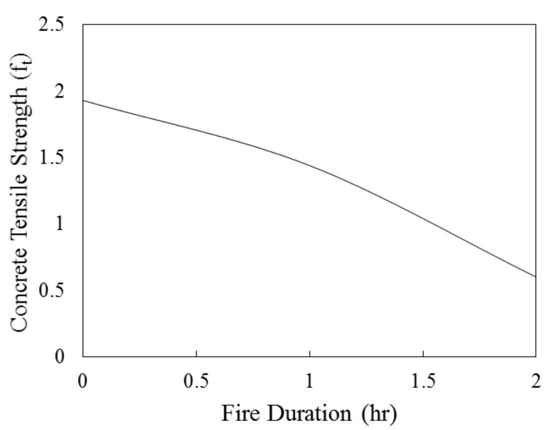


Figure 3.12 Reduction in concrete tensile strength during fire exposure.

3.3.1.3 Reduction of shear capacity during fire exposure

Figure 3.13 shows the reduction of the shear capacity of beam B501 during exposure to a fire as predicted by the MCFT. The shear capacity of the beam reached about 42% of its initial shear capacity at ambient temperature after 1.72 hr of fire exposure. Desai et al. (1998) reported that the beam failed after 1.78 hr of fire exposure while supporting a shear force of 60 kN. The proposed method predicted the beam to fail after 1.72 hr of fire exposure for the same shear force.

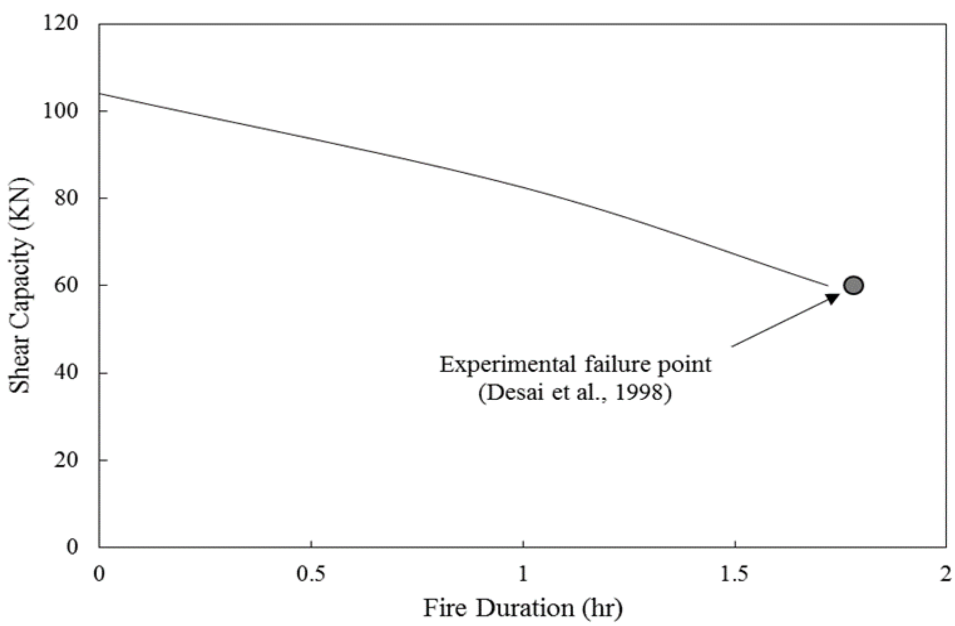


Figure 3.13 Reduction of Shear Capacity of B501 during fire.

3.3.1.4 Shear capacity predictions for the remaining beams

Figure 3.14 shows the reduction in the shear capacity of beams (B102, B202, B301, B401, and B501). Desai et al. (1998) reported that beams B102, B202, B301, B401, and B501 failed after 1.88, 1.68, 1.70, 1.76, 1.78 hr of fire exposure while supporting shear forces of 35, 40, 45, 55, and 60 kN, respectively. For the same shear forces, the proposed method predicted the beams to fail after 1.77, 1.55, 1.56, 1.75, 1.72 hr of fire exposure. Figure 3.15 presents a comparison between the predictions of the proposed method and the experimental results for all beams. It can be concluded from the figures that the proposed method could predict with sufficient accuracy the shear capacity of beams during exposure to a fire event.

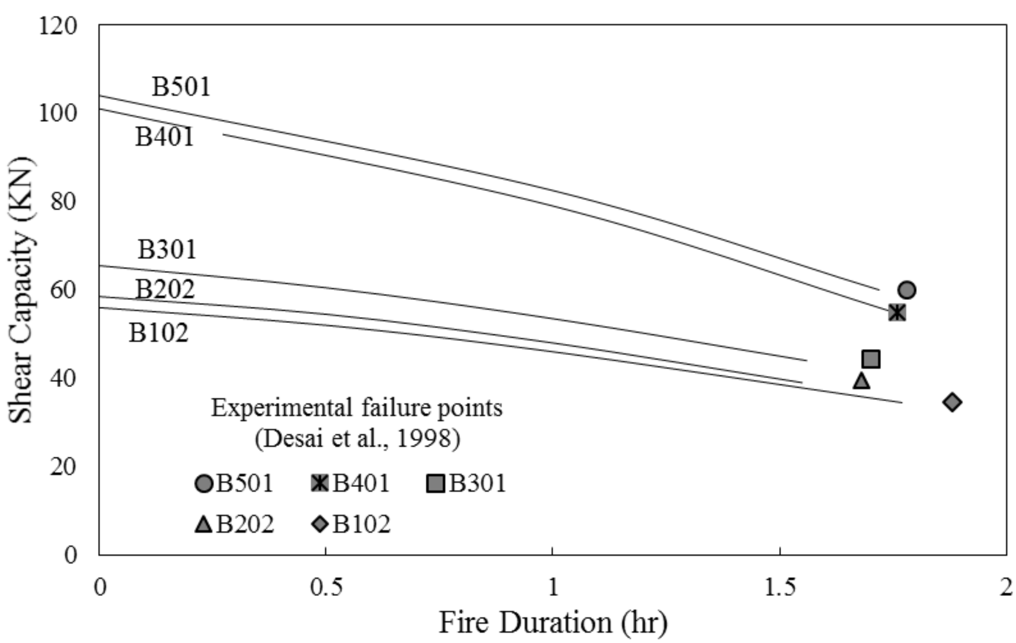


Figure 3.14 Shear capacity predictions using the proposed method.

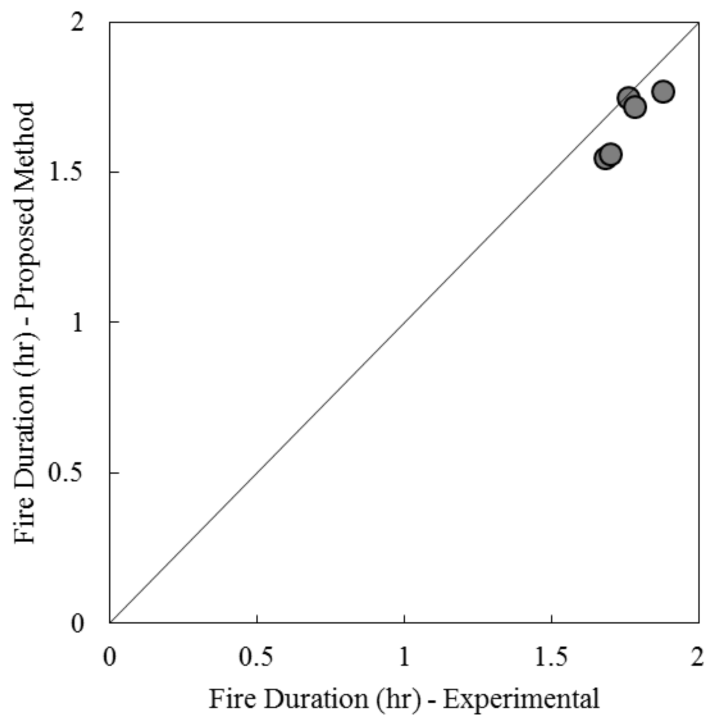


Figure 3.15 Comparison between the predictions of the proposed method with experimental results (Desai, 1998).

3.3.2 Hsu et al.'s analytical work

Hsu et al. (2008) developed an analytical model that incorporates thermal and structural analyses to assess the shear capacity of reinforced concrete beams during fire exposure. This model used the finite difference method to model the temperature distribution within the beam's cross-section. The shear capacity was evaluated using the American Concrete Institute (ACI) building code while considering the influence of elevated temperatures on the properties of steel and concrete. They applied the model on an assumed beam to study its shear strength during fire exposure. The beam had 300 x 500 mm cross-section size. It was reinforced with 4φ25 mm at its bottom and 2φ25 mm at its top. Stirrups of φ 10 mm spaced at 100 mm were used. The compressive strength of concrete was 20 MPa, and the yield strength of the steel bars was 400 MPa. The proposed method is applied to the same beam. Figure 3.16 shows a good match between the prediction of the proposed method and results obtained from Hsu's analytical model (Hsu et al., 2008).

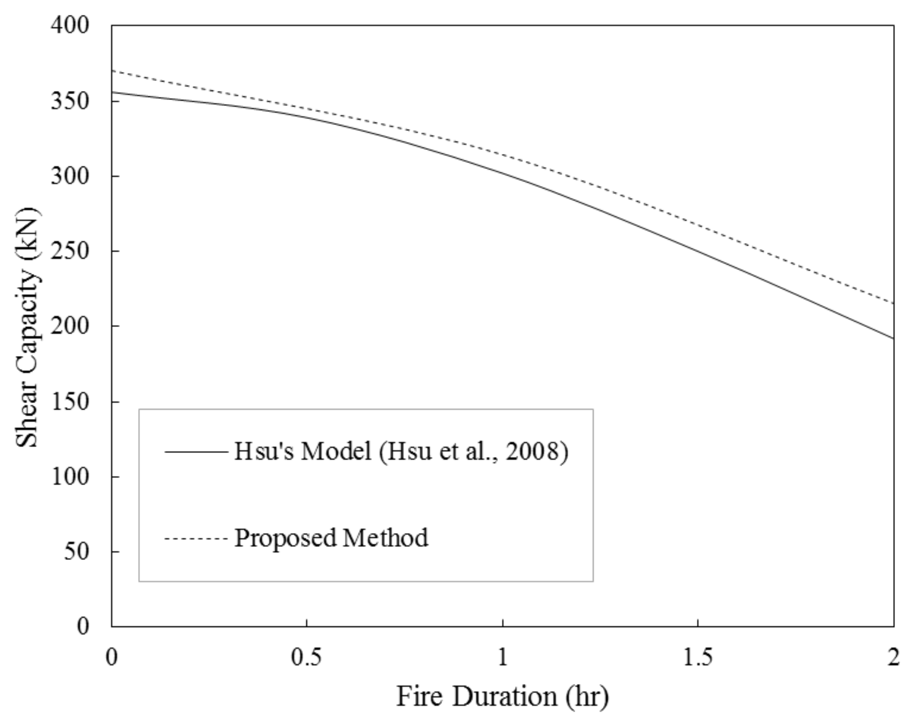


Figure 3.16 Comparison of shear capacity reduction during fire exposure between the proposed method and Hsu's model.

3.4 Parametric study

A total of 26 beams were analyzed, as shown in Table 3.3. Three beam heights (300 mm, 400 mm, and 500 mm), three beam widths (200 mm, 300 mm, and 400 mm), two longitudinal reinforcements ratios (1.5% and 2.5%), four transverse reinforcement ratios (0.0%, 0.2%, 0.4%, and 0.6%), two concrete covers (30 mm and 40 mm), two concrete compressive strengths (30 MPa and 50 MPa), and five fire durations (0.0 hr, 0.5 hr, 1.0 hr, 1.5 hr and 2.0 hr) were considered. These beams were subjected to fire from three sides. Fire temperatures were based on the standard ISO 840 fire curve (Lie, 1992).

Table 3.3 Parametric study beams

Beam #	f_c' (MPa)	b (mm)	h (mm)	ρ_l (%)	ρ_t (%)	Cover (mm)
B1	30	200	300	1.50	0.00	30
B2					0.20	
B3					0.40	
B4					0.60	
B5				2.50	0.20	
B6					0.40	
B7					0.60	
B8				1.50	0.20	40
B9					0.40	
B10					0.60	
B11			400		0.00	30
B12					0.20	
B13					0.40	
B14		300			0.00	
B15					0.20	
B16					0.40	
B17	50	200	300		0.00	
B18					0.20	
B19					0.40	
B20					0.60	
B21	30		500		0.00	
B22					0.20	
B23					0.40	
B24		400			0.00	
B25					0.20	
B26					0.40	

3.4.1 Parametric study results

The following sections provide a detailed analysis of the effect of each parameter considered in this study on the beam shear capacity during exposure to elevated temperatures.

3.4.1.1 Effect of the transverse reinforcement ratio (ρ_w)

Figure 3.17 tracks the shear capacity reduction, during exposure to elevated temperatures for beams with different percentage of transverse reinforcement. Figure 3.18 shows the relationship between the percentage of transverse reinforcement and the shear capacity for the beams at three fire exposure durations.

The reduction in the shear capacity after 1 hr and 2 hr of fire exposure is 22% and 57% for B2, and 26% and 60% for B4. These numbers suggest that the reduction rate of the shear capacity during fire is not significantly affected by the transverse reinforcement ratio. By increasing the transverse reinforcement ratio (ρ_t) from 0.2% to 0.4%, the shear capacity increased by 49% at ambient temperature, and by 40% and 37% after 1 hr and 2 hr of fire exposure, respectively. Similarly, by increasing the transverse reinforcement ratio (ρ_t) from 0.4% to 0.6%, the shear capacity is increased by 27% at ambient temperature, and by 24% and 20% after 1 hr and 2 hr of fire exposure, respectively. It can be noted that the benefit of using additional shear reinforcement decreases during fire because of the reduction of the steel yield strength.

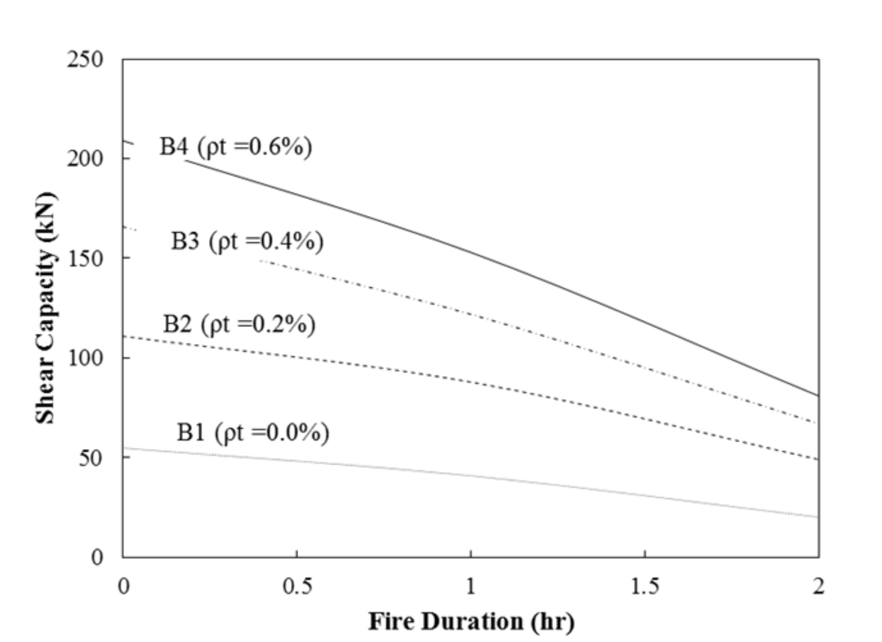


Figure 3.17 Reduction in shear capacity with fire duration for different web reinforcement percentages.

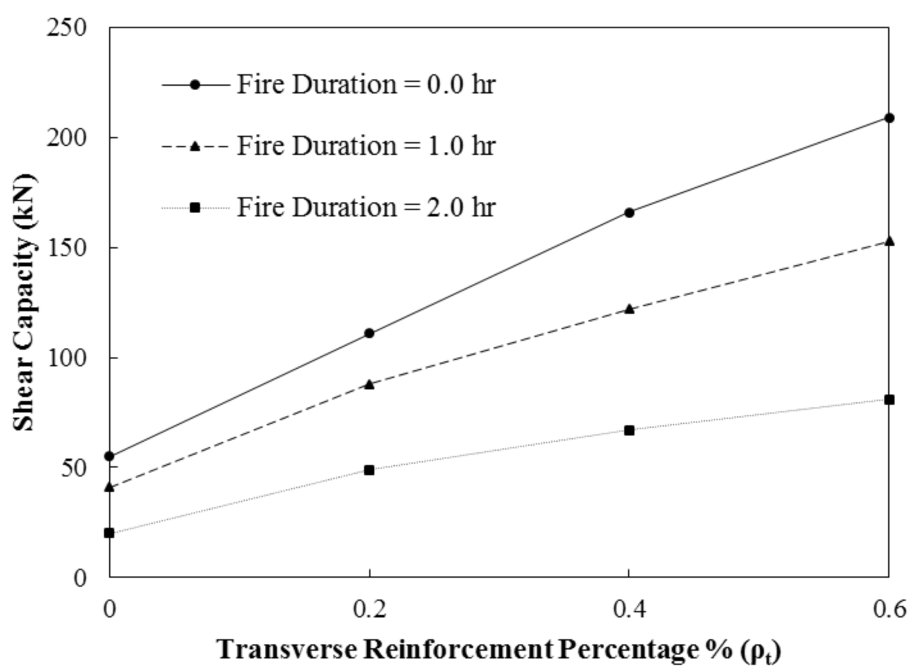


Figure 3.18 Effect of amount of web reinforcement on shear capacity during Fire.

3.4.1.2 Effect of the longitudinal reinforcement ratio (ρ_l)

Figure 3.19 tracks the shear capacity reduction, during exposure to elevated temperatures, of four beams B2, B4, B5, and B7 that have the same cross-section dimensions (200 x 300 mm). Beams B2 & B5 have same web reinforcement (0.2%), and two different percentages of longitudinal reinforcement (1.5% and 2.5%). Beams B4 and B7 have same web reinforcement (0.6%), and two different percentages of longitudinal reinforcement (1.5% and 2.5%).

For beams with 0.2% transverse reinforcement: The shear capacity of beam B2, which has a longitudinal reinforcement of 1.5%, is reduced by 22% and 57% after 1 hr and 2 hr of fire exposure, respectively. The shear capacity of beam B5, which has a longitudinal reinforcement of 2.5%, is reduced by 20% and 55% after 1 hr and 2 hr of fire exposure, respectively.

For beams with 0.6% transverse reinforcement: The shear capacity of beam B4, which has a longitudinal reinforcement of 1.5%, is reduced by 26% and 60% after 1 hr and 2 hr of fire exposure, respectively. The shear capacity of beam B7, which has a longitudinal reinforcement of 2.5%, is reduced by 24% and 59% after 1 hr and 2 hr of fire exposure, respectively.

Increasing the percentage of longitudinal reinforcement slightly increases the shear capacity of beams during fire. This increase in the percentage of longitudinal reinforcement does not affect the shear capacity reduction rate during exposure to a fire event.

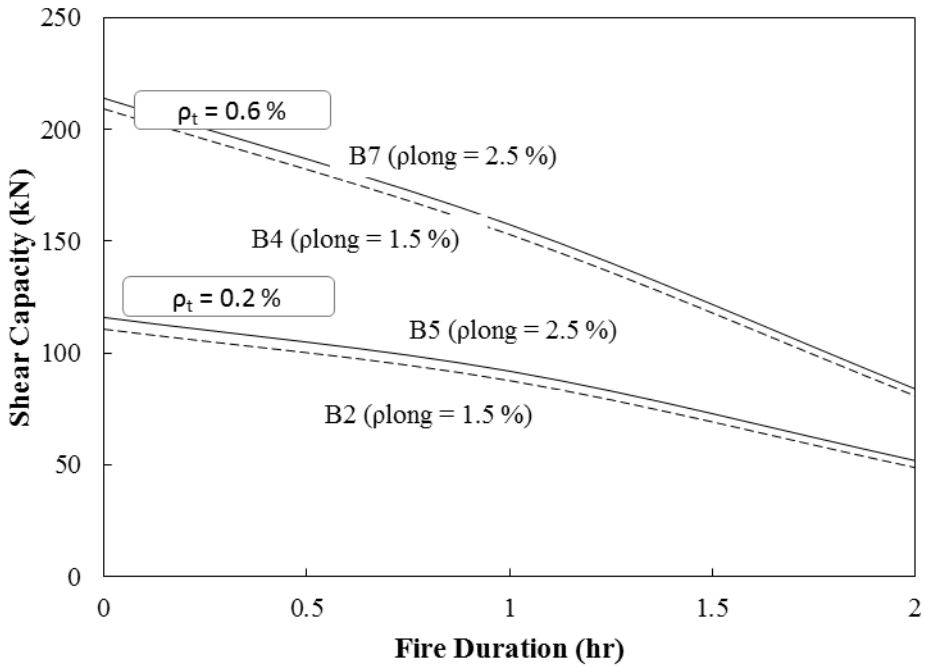


Figure 3.19 Reduction in shear capacity with fire duration for different longitudinal reinforcement percentages.

3.4.1.3 Effect of concrete cover

Figure 3.20 tracks shear capacity reduction, during exposure to elevated temperatures, of beams B2, B4, B8, and B10. Beams B2 & B4 have a 30 mm concrete cover, while beams B8 & B10 have a 40 mm concrete cover.

For beams with 0.2% transverse reinforcement: The shear capacity of beam B2, which has a 30 mm concrete cover, is reduced by 22% and 57% after 1 hr and 2 hr of fire exposure, respectively. On the other hand, the shear capacity of beam B8, which has a 40 mm concrete cover, is reduced by 14% and 44% after 1 hr and 2 hr of fire exposure, respectively.

For beams with 0.6% transverse reinforcement: The shear capacity of beam B4, which has a 30 mm concrete cover, is reduced by 26% and 60% after 1 hr and 2 hr of fire exposure, respectively. On the other hand, the shear capacity of beam B10, which has a 40 mm concrete cover, is reduced by 20% and 50% after 1 hr and 2 hr of fire exposure, respectively. Increasing the concrete cover from 30 mm to 40 mm increased the shear capacity by 8% and 24% after 1 hr and 2 hr of fire exposure, respectively.

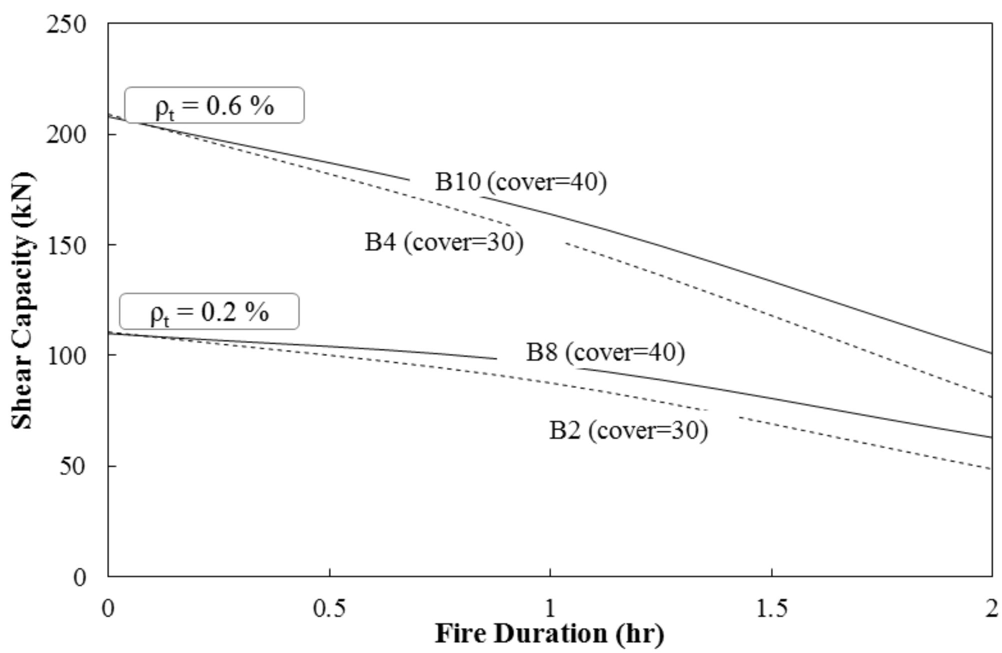


Figure 3.20 Reduction in shear capacity with Fire duration for different concrete covers.

3.4.1.4 Effect of beam height (h)

Figure 3.21 tracks the shear capacity reduction, during exposure to elevated temperatures of beams B1, B3, B11, B13, B21, and B23. Beams B1 & B3 have a 300 mm beam height and web reinforcement percentages of 0% and 0.4%, respectively. Beams B11 & B13 have a 400 mm beam height and web reinforcement percentages of

0% and 0.4%, respectively. Beams B21 & B23 have a 500 mm beam height and web reinforcement percentages of 0% and 0.4%, respectively.

For beams with no transverse reinforcement: The shear capacity of beam B1, which has a 300 mm beam height, is reduced by 25% and 64% after 1 hr and 2 hr of fire exposure, respectively. The shear capacity of beam B11, which has a 400 mm beam height, is reduced by 20% and 52% after 1 hr and 2 hr of fire exposure, respectively. The shear capacity of beam B21, which has a 500 mm beam height, is reduced by 18% and 48% after 1 hr and 2 hr of fire exposure, respectively.

For beams with 0.4% transverse reinforcement: The shear capacity of beam B3, which has a 300 mm beam height, is reduced by 26% and 59% after 1 hr and 2 hr of fire exposure, respectively. The shear capacity of beam B13, which has a 400 mm beam height, is reduced by 24% and 58% after 1 hr and 2 hr of fire exposure, respectively. The shear capacity of beam B23, which has a 500 mm beam height, is reduced by 22% and 56% after 1 hr and 2 hr of fire exposure, respectively.

Increasing the beam height results in a higher shear capacity of the beams. The shear capacity reduction rate during exposure to a fire event for beams with web reinforcement is not affected by increasing the beam height. This is due to the fact that increasing the beam height has a minor effect on the temperature of the web reinforcement. On the other hand, beams with no web reinforcement showed a slightly higher reduction rate of shear capacity in beams of smaller heights.

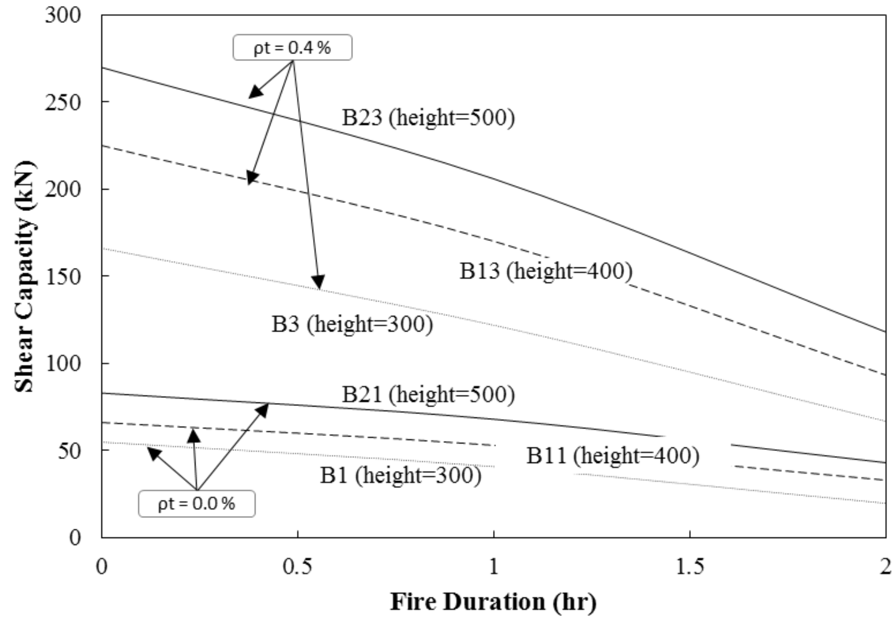


Figure 3.21 Reduction in shear capacity with fire duration for different beam heights.

3.4.1.5 Effect of beam width (b)

Figure 3.22 tracks the shear capacity reduction, during exposure to elevated temperatures, of four beams B21, B23, B24, and B26. Beams B21 & B24 have the same percentages of transverse reinforcement 0.0%, and two different widths 200 mm and 400 mm, respectively. Beams B23 & B26 have the same percentages of transverse reinforcement 0.4%, and two different widths 200 mm and 400 mm, respectively.

For beams with no transverse reinforcement: The shear capacity of beam B21, which has a 200 mm beam width, is reduced by 18% and 48% after 1 hr and 2 hr of fire exposure, respectively. On the other hand, the shear capacity of beam B24, which has a 400 mm beam width, is reduced by 8% and 16% after 1 hr and 2 hr of fire exposure, respectively.

For beams with 0.4% transverse reinforcement: The shear capacity of beam B23, which has a 200 mm beam width, is reduced by 23% and 56% after 1 hr and 2 hr of fire exposure, respectively. On the other hand, the shear capacity of beam B26, which has a 400 mm beam width, is reduced by 18% and 39% after 1 hr and 2 hr of fire exposure, respectively.

It can be seen that beams with larger width are less affected by fire hazards. Increasing the beam width is an effective way to maintain the shear capacity during fire as the concrete core is protected from fire temperature. Beams with large width and transverse reinforcement experienced higher reduction rates of shear capacity than beams with no transverse reinforcement. This is due to the fact that temperature of web reinforcement is not affected by the beam width.

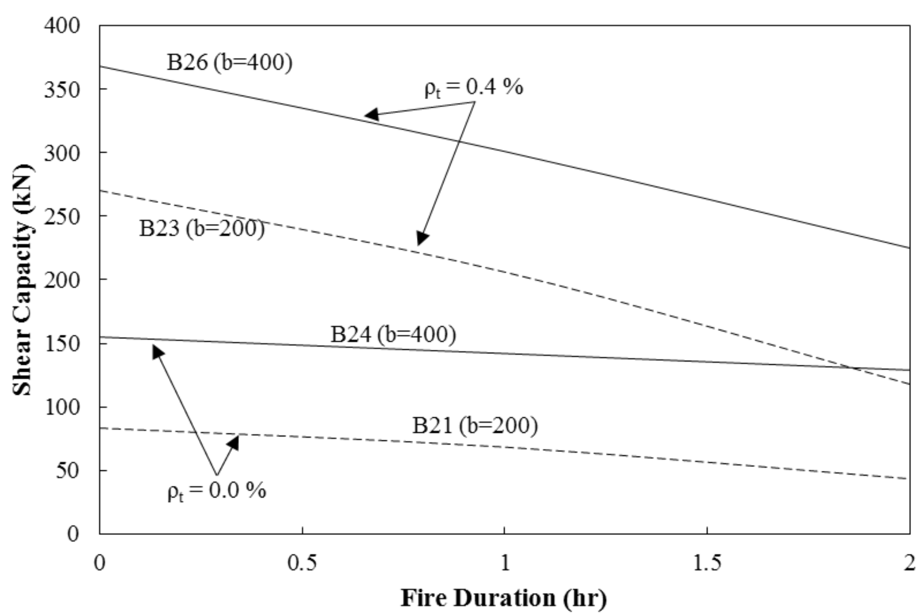


Figure 3.22 Reduction in shear capacity with fire duration for different beam widths.

3.4.1.6 Effect of concrete compressive strength (f_c')

Figure 3.23 tracks the shear capacity reduction, during exposure to elevated temperatures, of beams B1, B3, B4, B17, B19, and B20. To investigate the effect of concrete compressive strength (f_c'), the beams are divided into two groups: beams B1, B3, and B4 have a concrete compressive strength of 30 MPa, while beams B17, B19, and B20 have a concrete compressive strength of 50 MPa. Beams (B1 & B17), (B3 & B19), and (B4 & B20) have web reinforcement percentages of 0.0%, 0.4%, and 0.6%, respectively.

For beams with no transverse reinforcement: The shear capacity of beam B1, which has a 30 MPa compressive strength, is reduced by 25% and 64% after 1 hr and 2 hr of fire exposure, respectively. On the other hand, the shear capacity of beam B17, which has a 50 MPa compressive strength, is reduced by 23% and 57% after 1 hr and 2 hr of fire exposure, respectively. *For beams with 0.6% transverse reinforcement*: The shear capacity of beam B4, which has a 30 MPa compressive strength, is reduced by 26% and 60% after 1 hr and 2 hr of fire exposure, respectively. On the other hand, the shear capacity of beam B20, which has a 50 MPa compressive strength, is reduced by 27% and 63% after 1 hr and 2 hr of fire exposure, respectively.

For beams with no transverse reinforcement: By increasing the concrete compressive strength from 30 MPa to 50 MPa, the shear capacity is increased by 20% at ambient temperature and by 23% and 39% after 1 hr and 2 hr of fire exposure, respectively. *For beams with 0.6% transverse reinforcement*: By increasing the concrete compressive strength from 30 MPa to 50 MPa, the shear capacity is increased by 10% at ambient temperature and by 8% and 5% after 1 hr and 2 hr of fire exposure, respectively.

Using higher compressive strength ($f_c^{'}$) slightly increases the shear capacity of the beam. The effect of using higher concrete compressive strength becomes less pronounced for longer fire durations for beams with web reinforcement. This is due to the fact that web reinforcement has a greater contribution to the beam shear capacity, and at high temperatures the effect of the deterioration of web reinforcement overcomes the deterioration of concrete strength.

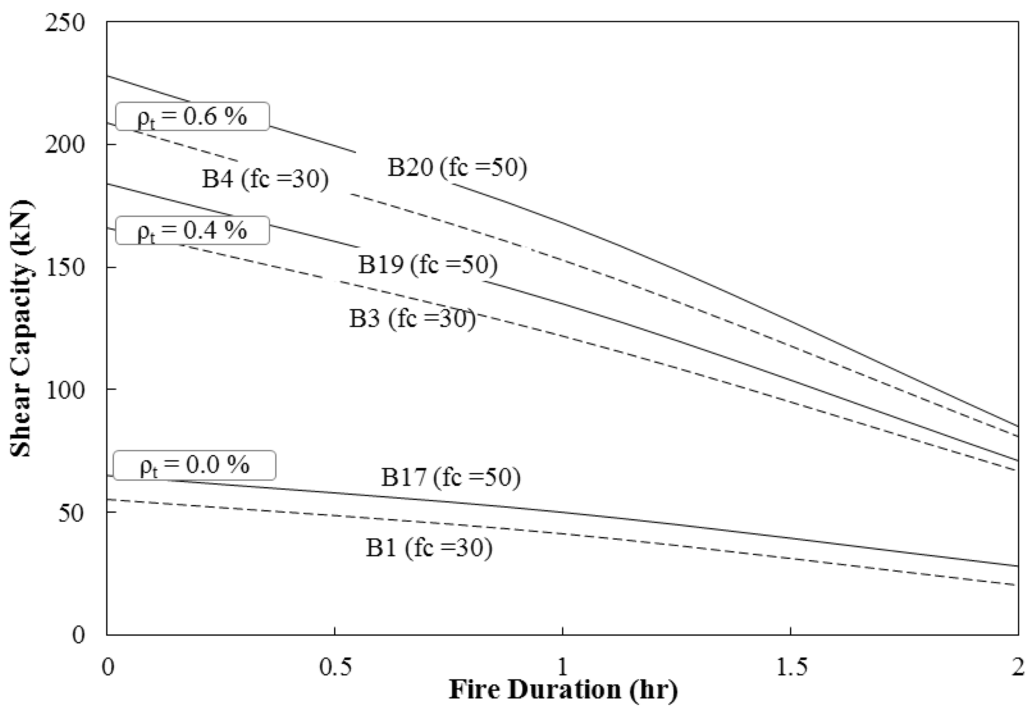


Figure 3.23 Reduction in shear capacity with fire duration for different concrete strengths.

3.5 Summary and conclusions

This chapter presented an analytical method to estimate the shear capacity of rectangular reinforced concrete beams exposed to elevated temperatures from three sides. The Finite Difference Method was implemented to perform the heat transfer analysis due to its simplicity and accuracy. Based on the results of the heat transfer analysis, average temperatures of concrete and shear reinforcement were calculated. The deteriorated concrete and steel strength properties were then calculated based on these average temperatures created within the concrete and steel. Shear capacity was subsequently estimated using the MCFT. The estimations of the proposed method were found to have a good agreement with the experimental and analytical studies found in the literature.

A parametric study was conducted to investigate the effects of different parameters on the shear capacity of reinforced concrete beams exposed to fire. The studied parameters were transverse reinforcement and longitudinal reinforcement ratio, concrete cover and strength, beam height and width, and fire duration. The effects of increasing concrete cover and increasing cross-section width were found to be the important factors that contribute towards reducing the fire damage on the shear capacity during exposure to elevated temperatures. Increasing the thickness of concrete cover is helpful to protect the damage of fire by delaying the effect of elevated temperatures on the strength of shear reinforcement. Increasing the cross-section width is helpful to reduce the damage of fire by protecting the concrete core from fire temperature.

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Chapter 4

4 Simplified practical method to predict the shear capacity of RC beams exposed to fire

This chapter aims to provide the design engineers with a simple tool to predict the shear capacity of RC beams exposed to fire conditions. The method begins with the development of simplified equations to calculate average temperatures of the shear reinforcement and the concrete cross-section. These calculated temperatures are then used to assess the corresponding deteriorated strength properties and to predict the shear capacity using the simplified formulas of the Canadian Standards Association (CSA A23.3-04). The proposed method is validated using the predictions of the analytical method presented in chapter (3) and experimental results by others. A set of simple design charts to determine the average temperature of shear reinforcement and concrete are also provided.

4.1 Introduction

Concrete structures are currently being designed for fire resistance using prescribed methods that are based on experimental fire tests. These prescribed methods involve specifying minimum cross-section size and minimum concrete cover (El-Fitiany and Youssef, 2009). As new codes are moving towards performance-based design, there is a need for design tools to assess the capacity of the various structural elements during fire conditions.

Capacity of reinforced concrete structural elements during fire can be assessed using analytical methods such as finite element method (FEM) and finite difference method (FDM). Raut and Kodur (2011) concluded that the FEM is an accurate tool to evaluate the behavior of the RC structural elements during fire exposure. However, applying the FEM in fire situations requires long processing times, and, thus it is not a practical tool for design engineers. Although the FDM is relatively simpler to apply when compared to the FEM, it requires knowledge of heat transfer principles to evaluate the temperature distribution within the concrete section in addition to the need to account for different constitutive relationships for the section layers (Law, 2010 and Caldas, 2010).

Simplified tools to assess the load capacity of RC beams exposed to fire are needed to facilitate the move to performance-based design. Previous research has focused on developing such tools considering the flexural and axial capacities of concrete elements during fire. El-Fitiany and Youssef (2009) developed and validated a sectional analysis method to predict the flexural and axial behavior of RC members exposed to fire conditions. El-Fitiany and Youssef (2014) utilized the sectional analysis method to analyze continuous RC beams exposed to fire. El-Fitiany (2013) presented simplified methods to construct the interaction diagrams of fire-exposed elements and to analyze RC frames exposed to fire.

Shear resistance, in contrast, has not been thoroughly investigated (Bamonte et al., 2009). This chapter, therefore, aims at developing a simplified method to predict the shear capacity of RC beams exposed to fire.

4.2 Proposed simplified method to predict the shear capacity of RC beams exposed to fire

The proposed method relies on the fact that the shear capacity for concrete sections exposed to elevated temperatures can be evaluated using the same procedures used at ambient temperatures while taking into account the reduced properties of concrete and steel (Fransen et al., 1995). Accordingly, the proposed method for predicting the shear capacity of RC beams exposed to fire conditions incorporates threes steps: (i) average temperatures of the concrete cross-section and of the shear reinforcement are first estimated, (ii) the deteriorated properties of materials are then calculated, (3) the shear capacity is predicted using the shear resistance formulas of CSA A23.3-04.

4.2.1 Average temperatures of shear reinforcement and of concrete cross-section

Conducting a comprehensive heat transfer analysis may not be feasible for practicing structural engineers. Hence, there is a need to develop a simple and reliable method to determine the elevated temperatures within a structural member's cross-section during exposure to fire. Such a method will facilitate the assessment of the deteriorated strength properties of materials and, consequently, the shear capacity of the considered beam can be predicted.

This section starts by summarizing Wickstrom's method (1986) to calculate the temperature distribution within a fire-exposed RC cross-section. The method is then utilized to develop new equations that predict the average temperatures of the shear reinforcement and of the concrete cross-section.

4.2.1.1 Wickstrom simplified formulas

A set of convenient formulas were developed and validated by Wickstrom (1986) to determine the temperature distribution within a RC cross-section exposed to fire. Wickstrom's formulas were based on the assumption that the RC element is exposed to the ISO 834 standard fire, and normal weight concrete is used in the element but can be easily utilized for other fire scenarios and concrete types.

The following steps summarize the application of Wickstrom's formulas to determine the temperature distribution within a RC element. Figure 4.1 shows an example of the application of Wickstrom's formulas on a RC beam that is exposed to fire from three sides; Left (L), Right (R), and Bottom (B) faces:

1) The fire temperature T_f (Celsius) is calculated at a specific fire duration t (hr) using a given fire temperature-time relationship.

2) A corresponding time of exposure to the standard ISO 834 standard fire, Eq. 4.1, is then calculated by determining a modified time (t^*). The ratio between the modified time (t^*) and the actual fire duration (t) defines a dimensionless compartment time factor (Γ).

$$T_{f\,(ISO)} = 345 \log(480 \mathrm{t}^* + 1) \tag{4.1}$$

where T_f (ISO) is the ISO 834 standard fire temperature in Celsius at a modified fire duration t^* in hrs.

3) The temperature rise at any point (x, y) inside the RC cross-section due to exposure to elevated temperatures can be calculated using the following equation.

$$T_{xy} = \left[\eta_w \left(\eta_x + \eta_y - 2\eta_x . \eta_y \right) + \eta_x . \eta_y \right] T_f \tag{4.2a}$$

$$\eta_w = 1 - 0.0616\ (\sqrt{\Gamma}.\ t)^{-0.88} \qquad \geq\ 0.0 \tag{4.2b}$$

$$\eta_x = \left[0.18 \ln \left(\frac{t}{x^2} \right) - 0.81 \right]_{Fire\ (L)} + \left[0.18 \ln \left(\frac{t}{(b-x)^2} \right) - 0.81 \right]_{Fire\ (R)} \quad \geq\ 0.0 \tag{4.2c}$$

$$\eta_y = \left[0.18 \ln \left(\frac{t}{y^2} \right) - 0.81 \right]_{Fire\ (B)} \qquad \geq\ 0.0 \tag{4.2d}$$

Where b is the beam width, h is the beam height, T_{xy} is the temperature rise at any point (x, y) in Celsius, η_w is the ratio between the surface temperature and the fire temperature, and η_x and η_y are the ratios between the internal and surface temperatures due to heating in the x and y directions, respectively.

4.2.1.2 Temperature regions

When a RC beam is exposed to elevated temperatures, such as a fire event from three directions (left, right, and bottom), the section can be divided into different temperature regions as shown in Figure 4.1. The temperature rise within each region will be different based on its distance from the heating surfaces. As illustrated in the figure, the values shown in each region indicate the heating surface that causes temperature variation in the x and y directions. It can be observed that region R4 (0, 0) is not affected by the fire temperature; however, region R2 (0, B) is affected by fire temperature from the bottom side. The values of x_v and y_v, which define the boundaries of these temperature regions, can be calculated by equating η_x and η_y in equations (2c) and (2d) to zero. Hence, values of x_v and y_v can be determined from the following equation:

$$x_v, y_v = \sqrt{\left(e^{-4.5} \; . \; t\right)} \qquad (4.3)$$

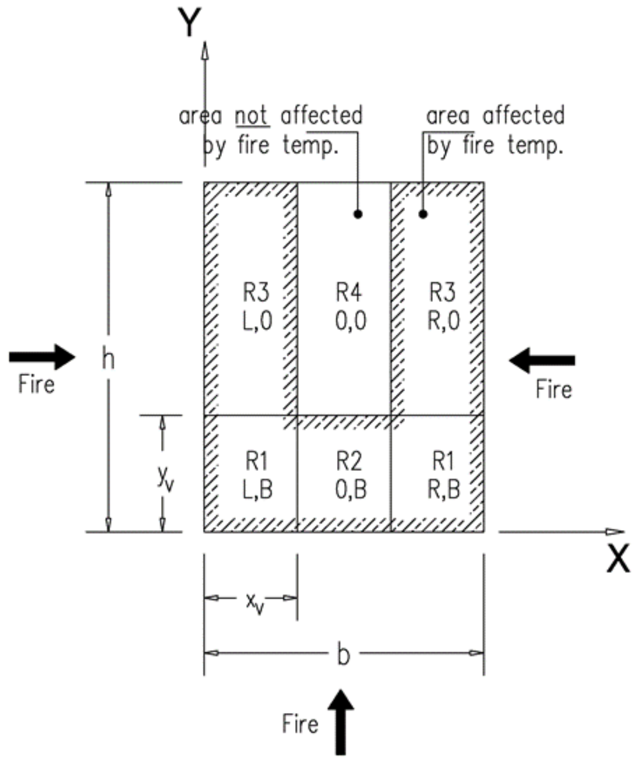


Figure 4.1 Temperature regions within a RC beam exposed to fire from three sides.

4.2.1.3 Average temperature of shear reinforcement

Equation (4.2) can be used to predict the temperature rise T_{xy} at different locations along the shear reinforcement. As the shear reinforcement is located at a fixed distance from the side of the beam, then value of *x* is constant. Hence, Equation (4.2) can be integrated with respect to *y*, from *c* to *h-c*, where *c* is the cover and *h* is beam height. The resulting value is then divided by the length of the shear reinforcement to get the average temperature at a specific fire exposure duration.

When this beam is exposed to fire, the temperature will rise depending on the number of the beam's faces exposed directly to the fire. Therefore, the beam cross-section will be divided into various temperature regions. As the beam is assumed to be exposed to fire from three sides, the shear reinforcement can be divided into two regions, as shown in Figure 4.2: (L1) which is affected by the fire temperature from x direction only (left or right), and (L2) which is affected by the fire temperature from x and y directions (left or right, and bottom). The boundary between those two regions is specified by value (y_v) which can be determined by Equation (4.3).

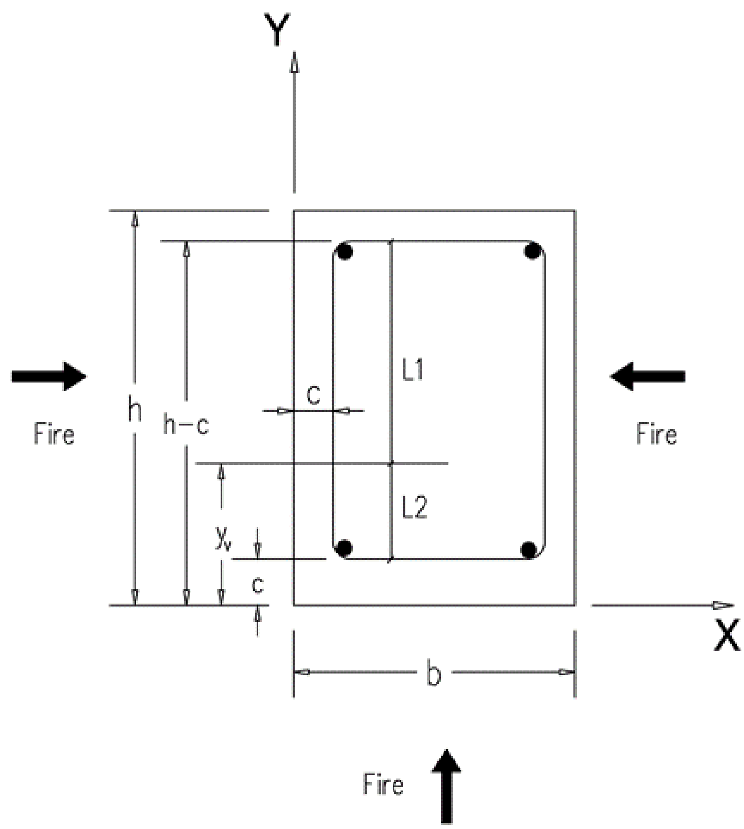


Figure 4.2 Temperature regions within the shear reinforcement.

Within region (L1) of the shear reinforcement the temperature rise is affected only by fire temperature in the x direction, with no effect from y-direction. This yields the following equation:

$$T_x = \eta_w . \eta_x . T_f \tag{4.4}$$

Within region (L2) of the shear reinforcement the temperature rise is affected by fire temperatures in the *x* and *y* directions. Hence, Equation (4.2) can be used to determine the temperature rise at any point within this region.

To get the average temperature within the web reinforcement, incorporating L1 and L2 regions, the following equation can be used:

$$Temp_{avg}(stirrups) = \frac{\int_c^{y_v} T_{xy}\ d(y) + T_x\ .\ (h - c - y_v)}{(h - c - c)} \tag{4.5}$$

Integrating the above equation results in Equation (4.6). This equation can be easily used by the practicing design engineers to get the average temperature in the shear reinforcement at each time step of a fire event. This can be achieved by direct substitution in the equation using the assumed time duration at which it is required to calculate the average temperature of the shear reinforcement *t*, along with values of *c* and *h* which are the cover and beam height, respectively.

<u>Average temperature of shear reinforcement</u>:

$$Temp\ (stirrups) = (A(y_v) - A(c) + B)/(h - 2c) \qquad (4.6)$$

Where

$$A(x) = -\{9x\ (4\,ln(10) + 69\,ln(480t + 1))\big(14630\,ln(t/c^2) + 20174\,ln(t/x^2) - 181250\,t^{(22/25)}\,ln(t/c^2) - 226250\,t^{(22/25)}\,\ln(t/x^2) - 2772\,ln(t/c^2)\,ln(t/x^2) + 1128125\ t^{(22/25)} + 22500\ t^{(22/25)}\,ln\,(t/c^2)\,ln(t/x^2) - 85085\big)\} / [1250000\,t^{(22/25)}\,\ln(10)]$$

$$B = (20 + 345\log(480\,t + 1))(1 - 0.0616\ t^{(-0.88)})(0.18\,ln\,(t/c^2) - 0.81))\ (h - c - y_v)$$

$$y_v = \sqrt{\frac{t}{90.02}}\ (m)$$

4.2.1.4 Average temperature of concrete cross-section

Similar to the above approach, the average temperature within concrete can be calculated using Equation (4.2). The difference; however, from the shear reinforcement, is that calculating the average temperature in the concrete cross-section requires performing double integration over the beam cross-section.

When a beam is exposed to fire from the three sides and $x_v \leq b/2$, as shown Figure 4.3 the beam cross-section will be divided into the following various temperature regions: (R1) which is affected by the fire temperature from *x* direction only (left or right), and

(R2) which is affected by the fire temperature from *y* direction only (bottom), (R3) which is affected by the fire temperature from *x* and *y* directions (left or right, and bottom), and (R4) which is not affected by fire. The boundaries among those regions can be specified by values (x_v, and y_v) that can be determined by Equation (4.3).

Within region (R1), the temperature rise is affected only by fire temperature in the *x* direction, with no effect from *y* direction, hence Equation (4.4) can be used in this region. Similarly, within region (R2), the temperature rise is affected only by fire temperature in the *y* direction, with no effect from y-direction, hence the following equation can be used in this region:

$$T_y = \eta_w . \eta_y . T_f \tag{4.7}$$

On the other hand, within region (R3), the temperature rise is affected by fire temperatures in the *x* and *y* directions, hence, Equation (4.2) will be used to determine the temperature rise at any point within this region.

To get the average temperature in the concrete, incorporating all regions, the following equation can be used:

$$Temp_{avg}(\text{Concrete}) = \frac{\int_0^{x_v} T_x \ d(x).(h - y_v).2 + \int_0^{x_v}\int_0^{y_v} T_{xy} d(y) \ d(x).2 + \int_0^{y_v} T_y \ d(y).(\frac{b}{2} - x_v).2}{(h \times b)} \tag{4.8}$$

When a beam is exposed to fire from the three sides and $x_v > b/2$, as shown in Figure 4.4, the beam cross-section will be divided into the following various temperature regions: (R1) which is affected by the fire temperature from x direction only (left or right), and (R2) which is affected by the fire temperature from both x directions (left and right) and y direction (bottom), (R3) which is affected by the fire temperature from x and y directions (left or right, and bottom), and (R4) which is affected by fire from both x directions. Equation (4.8) is applicable to represent this case as it accounts for the overlap within (R2) and (R4).

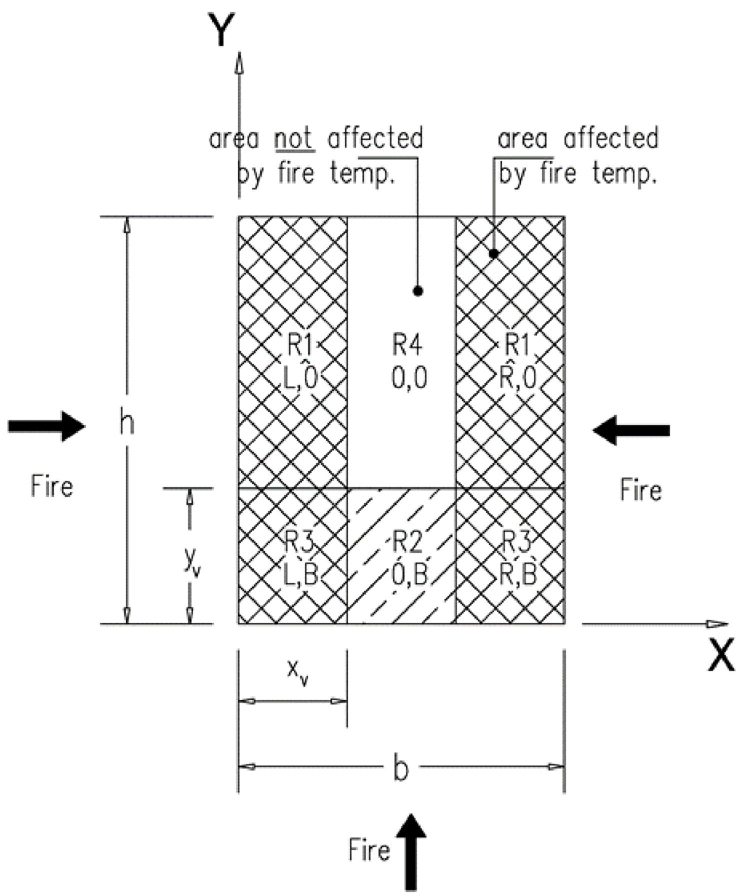


Figure 4.3 Temperature regions within concrete cross-section ($x_v \leq b/2$).

After integrating Equation (4.8), a simplified form is given by Equation (4.9). This equation can be easily used by the practicing design engineers to get the average temperature within the concrete cross-section at each time step of the fire event. This can be achieved by direct substitution in the equation using the assumed time duration at which it is required to calculate the average temperature of concrete *t*, along with values of *b* and *h*, which are the beam width and beam height, respectively.

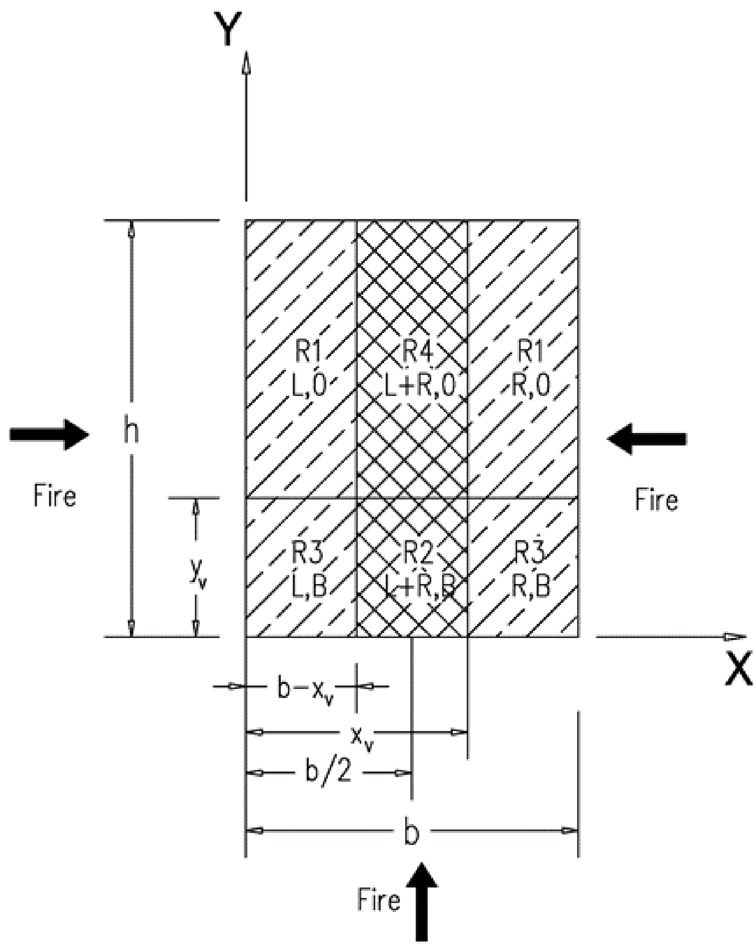


Figure 4.4 Temperature regions within concrete cross-section ($x_v > b/2$).

Average temperature of concrete cross-section:

$$Temp\ (concrete) = \frac{A(x_v)(h - y_v) + B + (0.5\,b - x_v)A(y_v)}{(b)(h)/2} \tag{4.9}$$

Where

$$A(x) = -(x(45(4\,ln(10) + 69\,ln(480t + 1))\left(1250\,t^{(22/25)} - 77\right) - 18\ ln(t/x^{\,2})(4\,ln(10) + 69\,ln(480t + 1))(1250\,t^{(22/25)} - 77)))/(25000\,t^{(22/25)}\,ln(10))$$

$$B = -(9x_v\,(6.4 \times 10^{15}\ln(480t + 1) + 8.6 \times 10^{14})(-55825y_v + 765625\,t^{(22/25)}\,y_v + 14630y_v\ln\left(t/x_v^{\,2}\right) + 14630\,y_v\ln(t/y_v^{\,2}) - 2772\,y_v\ln(t/x_v^{\,2})\ln(t/y_v^{\,2}) - 181250\ \ t^{(22/25)}\,y_v\ln\left(t/x_v^{\,2}\right) - 181250\ \ t^{(22/25)}\,y_v\ln(t/y_v^{\,2}) + 22500\ \ t^{(22/25)}\,y_v\ln(t/x_v^{\,2})\ \ln(t/y_v^{\,2})))/(2.7 \times 10^{20}\ \ t^{(22/25)})$$

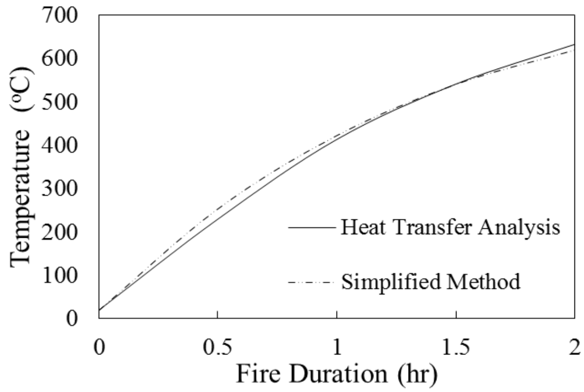
$x_v, y_v = \sqrt{\frac{t}{90.02}}$ (m)

4.2.1.5 Validation of the simplified equations to calculate the average temperatures

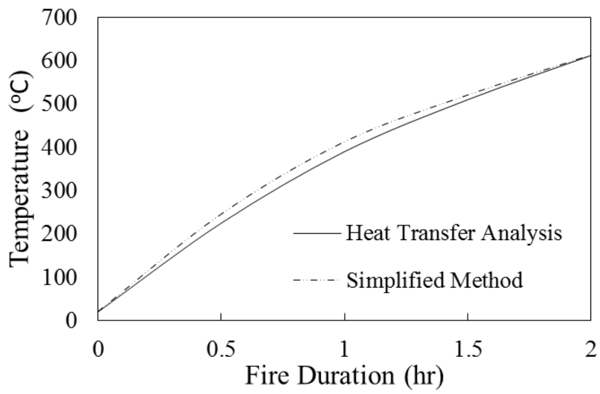
The developed equations, to calculate the average temperatures in the web reinforcement and in concrete, are validated by comparing their predictions with the results of the heat transfer analysis that was developed by El-Fitiany and Youssef (2009).

Figure 4.5 compares the average temperatures of the shear reinforcement, at different fire durations, predicted by the developed equations, with the average temperatures predicted by the heat transfer analysis for B1 (200 x 300 mm), B2 (400 x 500 mm), and B3 (600 x 700 mm). Similarly, Figure 4.6 compares the average temperatures of the concrete, at different fire durations, predicted by the developed equations, with the results obtained from the heat transfer analysis proposed by El-Fitiany and Youssef (2009), for beams B1, B2, and B3.

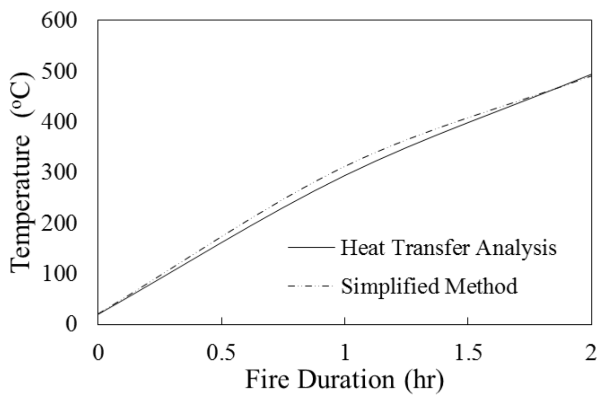
It can be observed from the figures that there is a good agreement between the estimates of the proposed equations and the heat transfer analysis at the different fire durations. Therefore, the proposed method, although simple, is found to be an accurate method to predict the average temperatures of the shear reinforcement and concrete, at different fire durations, when a beam is exposed to a fire. Hence, the proposed simplified approach can be used as a helpful and quick tool by design engineers due to its accuracy and simplicity of application.



a) B1

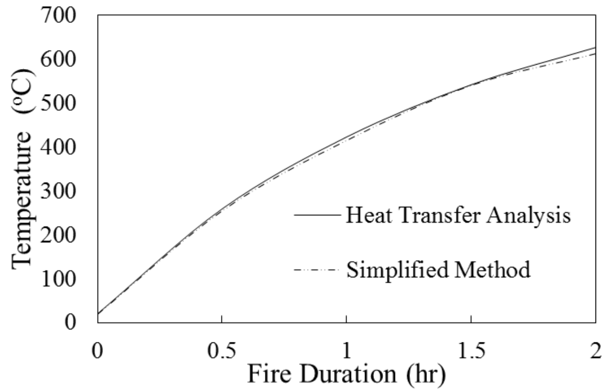


b) B2

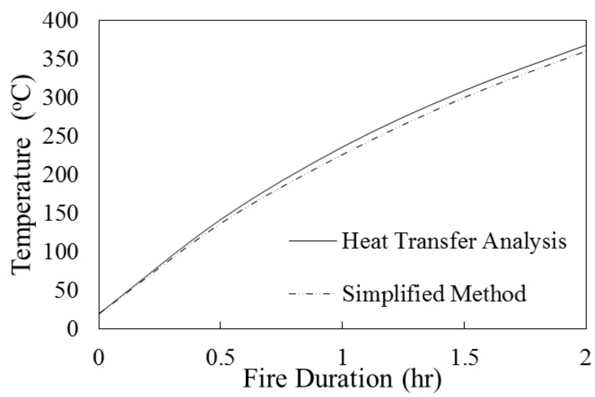


c) B3

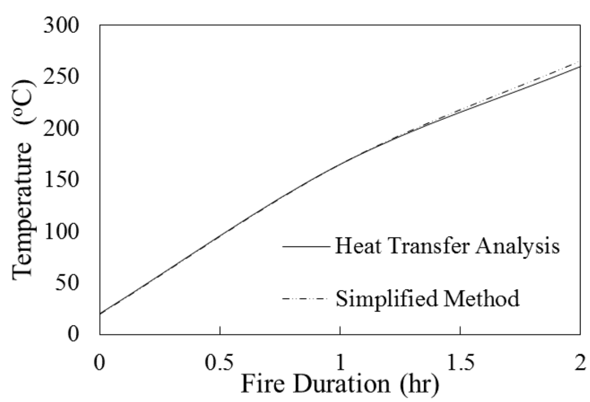
Figure 4.5 Average temperature of the shear reinforcement.



a) B1



b) B2



c) B3

Figure 4.6 Average temperature of concrete.

4.2.2 Properties of materials at elevated temperatures

Youssef and Moftah (2007) provided an assessment of the available models of the material properties of concrete and steel at elevated temperatures. Based on the recommendations of that study, the following models are used to evaluate the reduced strength properties of materials.

4.2.2.1 Concrete compressive strength

Hertz's model (Hertz, 2005) is used to predict the reduced concrete compressive strength at elevated temperatures (f'_{cT}):

$$f'_{cT} = \frac{f'_c}{1 + \dfrac{T}{15,000} + \left(\dfrac{T}{800}\right)^2 + \left(\dfrac{T}{570}\right)^8 + \left(\dfrac{T}{100,000}\right)^{64}} \quad (4.10)$$

Where (T) is the temperature in degrees Celsius, and (f'_c) is the concrete compressive strength at ambient temperature.

4.2.2.2 Yield stress of steel

Lie model (Lie, 1992) is used to predict the reduced yield strength of reinforcement steel at elevated temperatures (f_{yT}):

$$f_{yT} = \left[1 + \frac{T}{900 \ln(T/1750)}\right] \times f_y \qquad 0 < T \leq 600^o C \quad (4.11a)$$

$$f_{yT} = \left[\frac{340 - 0.34T}{T - 240}\right] \times f_y \qquad 600 < T \leq 1000^o C \quad (4.11b)$$

4.2.3 Prediction of shear capacity

As discussed in the previous sub-sections, the proposed simplified method to predict the shear capacity during fire exposure begins with using the new developed equations to determine the average temperatures of shear reinforcement and of concrete cross-section at different fire durations. The deteriorated strength properties of materials are then calculated based on these average temperatures. The final step of the proposed method is to estimate the shear capacity using the CSA A23.3-04 shear resistance formulas using the reduced strength properties of materials. Given the complexity of precisely determining the shear capacity of beams exposed to fire, one of the key benefits of the proposed method is that it is simple for practicing design engineers to apply.

4.2.4 Validation of the simplified method to predict the shear capacity during exposure to fire

The validation of the proposed simplified method is established by comparing its predictions with the experimental work by others and with the predictions of the analytical method that utilizes the MCFT presented in chapter (3). Figure 4.7 compares the shear capacity predictions of the proposed simplified method with the results of the experimental work by Desai et al. (1998) which is detailed in chapter (3). Figure 4.8 compares the shear capacity predictions of the proposed simplified method with results obtained from Hsu’s analytical model (Hsu et al., 2008) which is detailed in chapter (3). As shown in the figure, a good agreement can be noticed between the predictions of the proposed simplified method and the experimental and analytical results.

Figure 4.9 compares the shear capacity predictions for beam B1 using the simplified method with the shear capacity predictions that are obtained from utilizing the MCFT. Beam B1 has 200 x 300 mm cross-section, with φ7@200 web reinforcement. Strength of concrete ($f_c^{'}$) is 30 N/mm^2, and strength of steel (f_y) is 460 N/mm^2. Figure 4.10 compares the shear capacity predictions for beam B2 using the simplified method with the shear capacity predictions that are obtained from utilizing the MCFT. Beam B2 has 400 x 500 mm cross section, with φ7@200 web reinforcement. Strength of concrete ($f_c^{'}$) is 30 N/mm^2, and strength of steel (f_y) is 460 N/mm^2.

Figure 4.11 compares the shear capacity predictions for beam B3 using the simplified method with the shear capacity predictions that are obtained from utilizing the MCFT. Beam B2 has 600 x 700 mm cross section, with φ10@200 web reinforcement. Strength of concrete ($f_c^{'}$) is 30 N/mm^2, and strength of steel (f_y) is 460 N/mm^2. It can be observed from the figures that the results of the proposed simplified method have good matching with the results of the MCFT given the complexity of the problem.

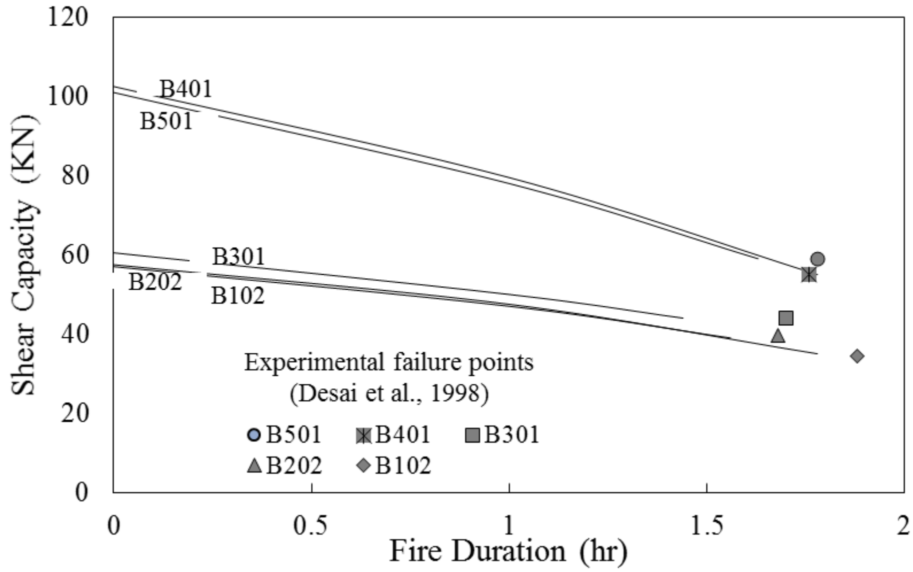


Figure 4.7 Comparison of the shear capacity using the simplified method with the experimental results (Desai et al., 1998).

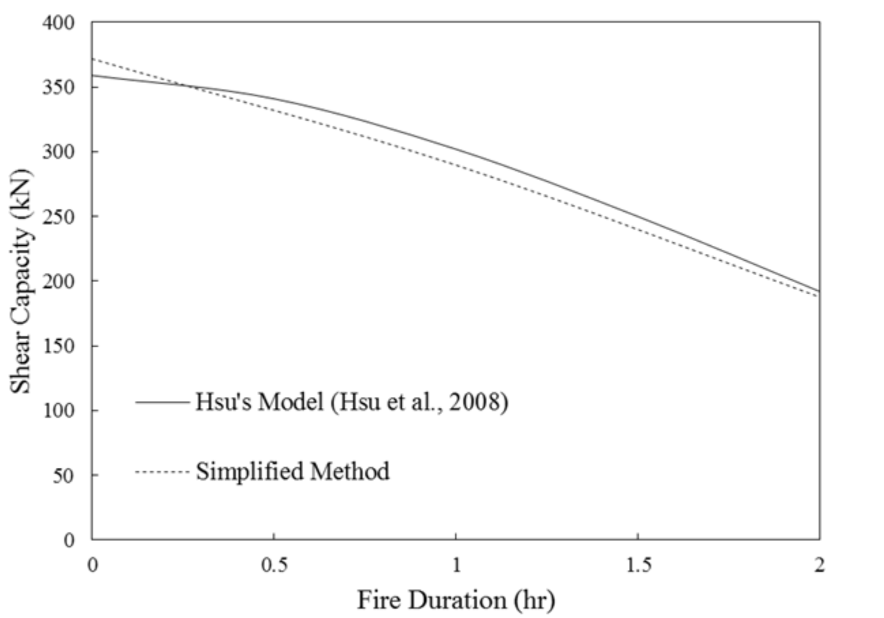


Figure 4.8 Comparison of shear capacity using the simplified method with Hsu's model (Hsu et al., 2008).

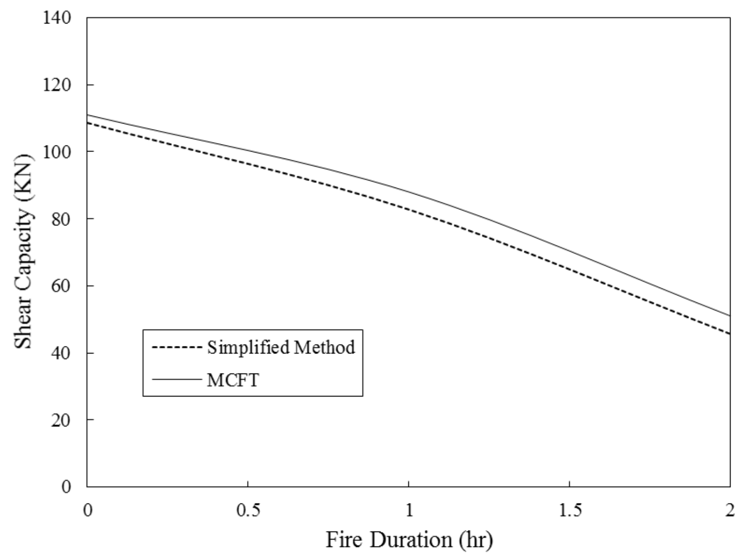


Figure 4.9 Comparison of the shear capacity of beam B1 using the simplified method with the shear capacity using MCFT.

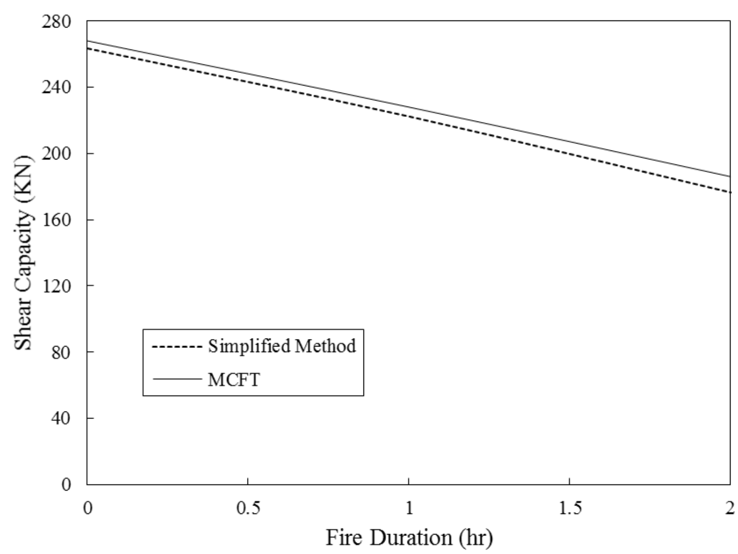


Figure 4.10 Comparison of the shear capacity of beam B2 using the simplified method with the shear capacity using MCFT.

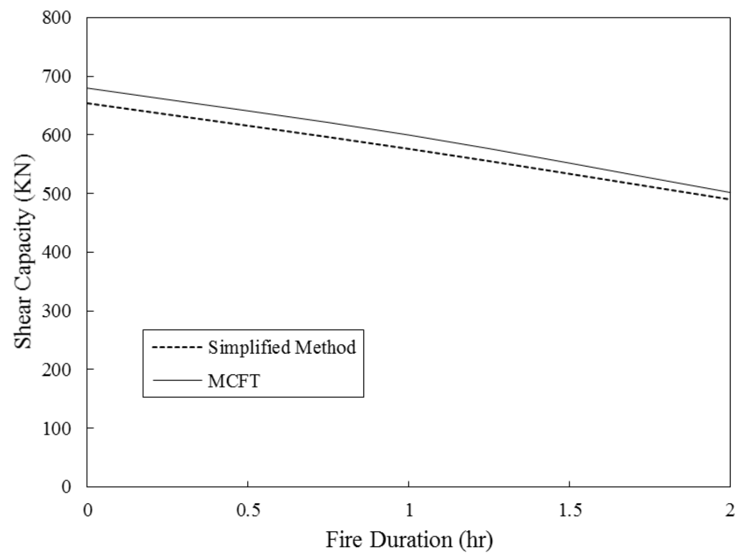


Figure 4.11 Comparison of the shear capacity of beam B3 using the simplified method with the shear capacity using MCFT.

4.3 Design charts to determine the average temperatures of stirrups and concrete

The simplified equations to determine the average temperatures of stirrups and the concrete are used to develop design charts that can be used by the design engineers to facilitate calculation of the average temperature of shear reinforcement and concrete. Figure 4.12 shows the relationship between the cover and the average temperature of stirrups for beam heights 300 mm and 700 mm. It can be seen from the figure that increasing the cover thickness provides an effective way to inhibit temperature increase in stirrups during exposure to fire and, hence, delaying the effect of elevated temperatures on the strength of shear reinforcement. Figure 4.13 shows the relationship between the beam width and the average temperature in concrete for different beam heights. It can be seen that increasing the cross section width provides an effective way to inhibit temperature increase in concrete during exposure to fire.

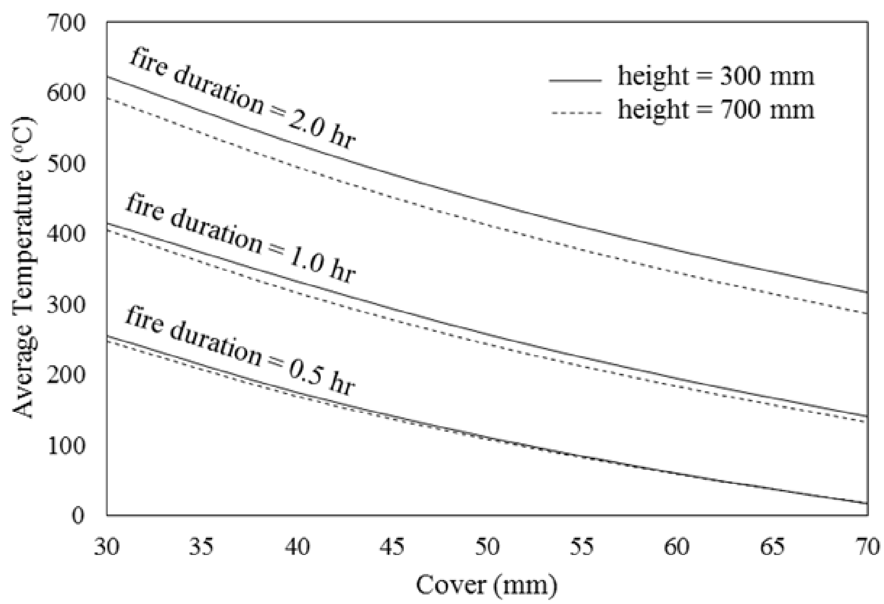
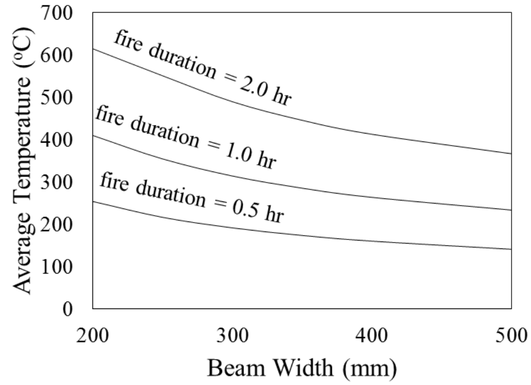
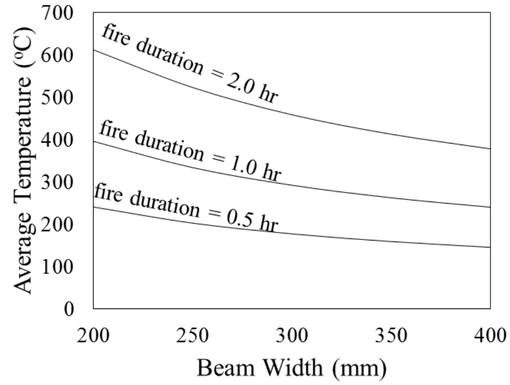


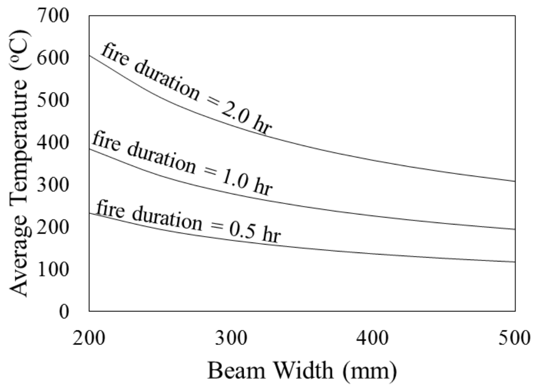
Figure 4.12 Design chart to determine the average temperature of stirrups during exposure to fire.



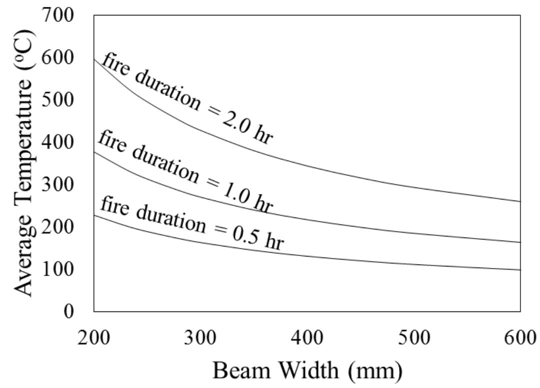
a) height = 300 mm



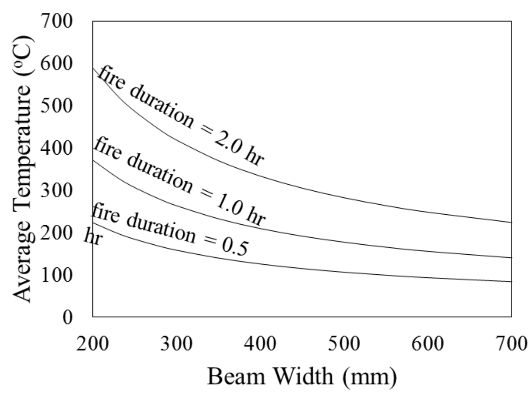
b) height = 400 mm



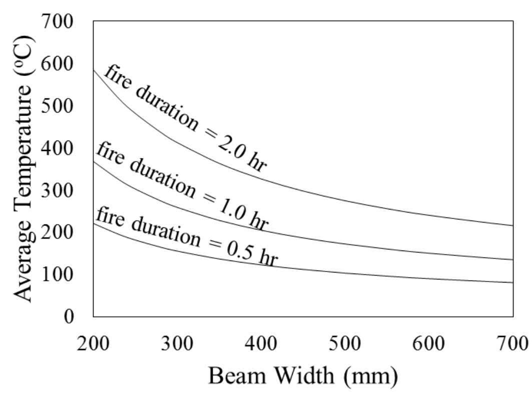
c) height = 500 mm



d) height = 600 mm



e) height = 700 mm



f) height = 800 mm

Figure 4.13 Design charts to determine the average temperature of concrete during exposure to fire.

4.4 Summary and conclusions

This chapter presented a simple and practical tool that can be used by structural engineers to predict that shear capacity of reinforced concrete beams exposed to fire from three sides. The proposed method assumes that the shear capacity for concrete sections exposed to elevated temperatures can be evaluated using the same procedures used at ambient temperatures while taking into account the reduced properties of concrete and steel.

The method utilizes new developed simple equations to calculate average temperatures of the shear reinforcement and the concrete cross section to facilitate the assessment of the deteriorated strength properties of materials. The analyzed sections were first divided into different temperature zones. Equations to evaluate the average temperature of stirrups and the average temperature of concrete were developed. A set of simple design charts to determine the average temperature in stirrups and concrete are also provided. The deteriorated strength properties of the materials are then calculated using these average temperatures. The shear capacity can be then estimated using the simple formulas of CSA A23.3-04 based on the reduced strength properties of the materials. The proposed simplified method was validated by comparing its predictions with experimental results by others and by results of the analytical method that involves using the MCFT.

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Chapter 5

5 Summary and conclusions

Concrete is one of the most commonly used construction materials due to its excellent properties including good fire resistance. However, concrete resistance to fire should not be taken for granted. Fire safety is an important design aspect that ensures structural integrity of reinforced concrete (RC) buildings during fire events. As new codes are moving from prescriptive methods towards performance based methods, design engineers are in need of rational design tools to assess the capacity of RC elements during a fire event. Previous research work focused on the flexural and axial capacities of concrete elements exposed to fire and there is a lack of research that assesses the shear capacity of concrete elements exposed to fire.

Basics of fire safety, different methods for evaluating the fire resistance of RC elements, and the need for rational tools to evaluate shear capacity during fire were discussed in the second chapter of this thesis. An analytical method to predict the shear capacity of RC beams exposed to elevated temperatures was developed and validated in the third chapter. Moreover, a parametric study to investigate the effects of different parameters on the shear capacity during exposure to fire was conducted. A simplified practical tool to predict the shear capacity of RC beams exposed to elevated temperatures was developed and validated in the fourth chapter. The following sub-sections summarize the work done in each chapter. This will be followed by the author's recommendation for further improvement and extension of the work done in this research.

5.1 Fire safety and the need for tools to evaluate shear capacity during exposure to fire

The basics of fire safety and the different methods for evaluating the fire resistance of RC structural elements were presented. Since the experimental methods and the advanced numerical simulations are expensive and time consuming, design engineers are in need of rational design tools to assess the capacity of RC elements during a fire event. The key findings of previous research work on the fire response of RC elements were summarized. It was observed that the previous research work has focused on the flexural and axial performance of structural elements during exposure to fire. Shear resistance, in contrast, has not been thoroughly investigated. The current codes of practice and standards are based on using prescribed methods for fire safety design of reinforced concrete structures. As new codes are moving towards performance-based design, there is a need for rational tools to evaluate the shear capacity during exposure to fire.

5.2 Analytical prediction of the shear capacity of RC beams at elevated temperatures

An analytical method to predict the shear capacity of RC beams exposed to elevated temperatures was proposed. The proposed method assumes that shear capacity can be derived using existing ambient temperature methods while accounting for the effect of elevated temperatures on the materials. The Finite Difference Method was implemented to perform the heat transfer analysis. Based on the results of the heat transfer analysis, average temperatures of concrete and shear reinforcement were calculated. The deteriorated concrete and steel strength properties were then calculated based on these elevated temperatures. Shear capacity was subsequently estimated using the MCFT. The estimations of the proposed method were found to have a good agreement with the experimental results found in the literature.

A parametric study was conducted to investigate the effects of different parameters on the shear capacity of RC beams exposed to fire. The studied parameters were transverse reinforcement and longitudinal reinforcement ratio, concrete cover and strength, beam height and width, and fire duration. The effects of increasing concrete cover and increasing cross section width were found to be the most important factors that contribute towards reducing the fire damage on the shear capacity during exposure to elevated temperatures. Increasing the thickness of concrete cover is helpful to protect the damage of fire by delaying the effect of elevated temperatures on the strength of shear reinforcement. Increasing the cross section width is helpful to reduce the damage of fire by protecting the concrete core from fire temperature.

5.3 Simplified practical method to predict the shear capacity of RC beams at elevated temperatures

A simplified practical tool that can be used by design engineers to predict the shear capacity of RC beams exposed to elevated temperatures was developed. The method utilizes new developed simple equations to calculate average temperatures of the shear reinforcement and the concrete cross section to facilitate the assessment of the deteriorated strength properties of materials. The analyzed section was first divided into different temperature zones. Equations to evaluate the average temperature of stirrups and the average temperature of concrete were developed.

The deteriorated strength properties of the materials were calculated using these average temperatures. The shear capacity was then predicted using the simplified formulas of the Canadian Standards Association (CSA A23.3-04). The proposed method was validated using the experimental results by others and the predictions of the analytical method that utilizes MCFT. A set of simple design charts to determine the average temperature of shear reinforcement and concrete were also provided. Increasing cover thickness and cross section width were found to provide an effective way to inhibit temperature increase during exposure to fire. The proposed method for predicting the shear capacity of RC beams during fire exposure is simple and practical, and so can readily be used by structural engineers.

5.4 Recommendations for future work

The following recommendations may be considered as an extension to the research conducted in this thesis:

- Develop tools to evaluate the effect of fire on shear capacity of prestressed members.
- Conduct an extensive experimental program to provide more validation for the use of the developed tools to predict the shear capacity at elevated temperatures.
- Study the shear strength of high strength concrete at elevated temperatures.
- Extend the use of the proposed method to study different cross section shapes and continuous RC beams.
- Extend the use of the proposed method to study the shear capacity of slabs and columns.

- Develop Finite Element Method to model the shear behavior at elevated temperatures.

5.5 References

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Curriculum Vitae

Name:	Mohamed Diab
Post-secondary Education and Degrees:	Cairo University Cairo, Egypt 1997-2002, B.Sc. The American University in Cairo Cairo, Egypt 2004-2008, M.E.Sc.
Honours and Awards:	Distinguished undergraduate student, Cairo University 1998-2002 Graduate Student Scholarship, The American University in Cairo 2005-2008 Western Graduate Research Scholarship, Western University 2011-2013
Memberships	ACI student membership ASCE student membership Engineers Syndicate of Egypt

Related Work Experience	Structural Engineer Dar Al-Handasah, Cairo, Egypt 2004-2011
	Teaching and Research Assistant Western University, Ontario, Canada 2011-2013