

ADDIS ABABA UNIVERSITY
ADDIS ABABA INSTITUTE OF TECHNOLOGY
SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING



**SENSITIVITY STUDY OF REINFORCED CONCRETE
BEAM EXPOSED TO HIGH TEMPERATURE: FINITE
ELEMENT MODEL**

A THESIS IN STRUCTURAL ENGINEERING

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A Thesis

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Degree of Master of Science**

The undersigned have examined the thesis entitled ‘SENSITIVITY STUDY OF REINFORCED CONCRETE BEAM EXPOSED TO HIGH TEMPERATURE: FINITE ELEMENT MODEL’ presented by **ALAYU BEFEKADU**, a candidate for the degree of **Master of Science**, and hereby certify that it is worthy of acceptance.

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UNDERTAKING

I certify that the research work titled "SENSITIVITY STUDY OF REINFORCED CONCRETE BEAM EXPOSED TO HIGH TEMPERATURE: FINITE ELEMENT MODEL" is my work. The work has not been presented elsewhere for assessment. Where material has been used from other sources it has been properly acknowledged/referred.

ALAYU BEFEKADU

ABSTRACT

Reinforced concrete structures significantly weakened their strength as a result of the fire. The behavior of concrete and RC-reinforced concrete elements at high temperatures has been extensively studied experimentally and analytically. This research paper, "Sensitivity Study of Reinforced Concrete Beam Exposed to High Temperature: A Finite Element Model", analyzed using the commercial software ABAQUS, refers to a three-dimensional (3D) nonlinear transient thermo-mechanical element (FE) analysis.

The purpose of this research is to find out the effect of fire on concrete cover, the compressive strength of concrete, the intensity of the fire, and the duration of the fire, as well as the load arising from the fracture, to understand the effects of fire.

Methodologically Numerical model simulation was employed with the aid of ABAQUS Software, which operated based on a finite element algorithm. The models were developed to analyze and understand the behavior of concrete and reinforced concrete beams with different intensities of fire, concrete cohesion, and compressive strength under different conditions.

The analysis result showed that when the temperature is from 200 °C to 700 °C ($^{\circ}\text{C}$ is a degree Celsius), the failure load decreases while the temperature increases. On the other hand, the concrete cover increased from 15mm to 25 mm, significantly increasing the failure load.

Generally, the results obtained from the nonlinear analyses of reinforced concrete beams under high-temperature duration and intensity are more sensitive compared to compressive strength and concrete cover. In addition, compared with the numerical and experimental solutions available in the literature, they were highly satisfactory.

Keywords: - Reinforced Concrete Beam, Concrete Cover, Compressive Strength, Fire Intensity Fire Duration, Sensitive Parameter, and Failure Load.

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Finally, but most definitely not least, I want to express my gratitude to my friends and family for their tremendous support.

List of Symbols/Acronyms

2D	two-dimensional
3D	three-dimensional
Fck	cube compressive strength
°C	Degree Celsius
E	modulus of elasticity
E _c	concrete modulus of elasticity
FEM	finite element modeling
CC	Concrete cover
Mpa	mega Pascal
% age	percentag
EC2	European standard code 2
λ	Thermal conductivity
C_p	Specific heat
Density	Density
CC, peak	specific heat capacity of concrete
RC	Reinforced concrete
L	Length
B	width
D	depth
$\epsilon\sigma$	stress-induced deformation
ϵ_{th}	free thermal deformation

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CHAPTER 1 INTRODUCTION

1.1 Background of the Study

Failure of a reinforced concrete structure due to design error, faulty construction process, foundation failure, extraordinary loads, unexpected failure modes, or any number of causes. However, failures are also caused by random special loads such as earthquakes, wind, and fire. A structure fails due to the strength dimension of the design material, and the design load on any structure is stressed beyond its strength limit, causing failure or excessive deformation.

One of the phenomena that can occur in any service structure at any time is a fire accident. It can be deliberately started to target enemies within society, such as protests that result in property loss, casualties, and disruptions to building access. It can also result from acts of terrorism, which are increasingly common these days, or from hostilities between enemies. Buildings can catch fire at any time, so protecting the structural integrity of the building and the safety of its people is crucial.

Little sources of fire provide the most harm, and many incidents are unpredicted. Interest in building structures that are fire-resistant has increased due to a number of dangerous and frequent fire incidents that have resulted in the loss of important human life, the failure of structures, and the challenging circumstances surrounding the quick containment of fire incidents. One of the most significant elements that can seriously harm buildings and structures is fire or elevated temperatures; therefore, in order to detect this effect, one must be aware of the failure parameter. Since fire is one of the most extreme climatic situations that a structure may encounter during its service life, fire safety precautions for structural components are an essential part of the design. Buildings exposed to temperature problems face structural integrity as their last line of defense when other fire control measures fail, which is the basis for this requirement.

Buildings must have adequate structural fire resistance to withstand such circumstances, or at the very least give occupants time to escape before strength and/or stability failure ensues. Because concrete is not merely a composite material, its composition has distinct properties, and its properties also depend on moisture and porosity, its thermal

Performance is more complex than that of most other materials. Concrete's mechanical and physical qualities will change when it is exposed to high temperatures. Reliable design evaluation and evaluation require a thorough understanding of the behavior of concrete under design-based accident conditions, both during and after prolonged exposure to high temperatures and thermal excursions. The characteristics of concrete are subject to alteration with time and environmental exposure, so it's critical to evaluate the effects of aging concrete while evaluating safety. The performance of specific structural members affected by high temperatures, code and design considerations for reinforced concrete structures exposed to high temperatures, and an examination of the effects of elevated temperatures on concrete materials are all covered in [30].

Compared to other building materials, concrete behaves fairly well in the event of a fire or high temperatures. Nevertheless, following fire exposure, its initial performance was negatively impacted. In addition to the original concrete's properties, the length and intensity of the fire also affect the reduction of mechanical resistance, elastic modulus, and fracture energy. Furthermore, concrete can break when exposed to extreme temperatures. The temperature inside the cross-section is significantly lower than the surface, which will heat up quickly when exposed to high temperatures. Concrete cracks may result from confined strains brought on by these temperature gradients [29].

Because the thermal expansion of the concrete varies from that of the steel bar, there is an extra effect in reinforced concrete. While the thermal expansion of concrete and reinforcing steel is roughly equal up to 400 oC, at higher temperatures there is a noticeable difference. Furthermore, steel's strength will diminish with temperature; nevertheless, in contrast to concrete, steel's strength nearly always returns after cooling. Numerous studies have tested the effects of high temperatures and flames on the behavior of concrete's structural qualities. The majority of mechanical strength tests are carried out after cooling, that is, to examine residual performance, due to the challenge of measuring at high temperatures [26–27].

Considering the kind of structure, the loading mechanism, and the nature of the fire, Building regulations' criteria for fire resistance are occasionally disregarded in structural design, which can result in expensive errors. Preliminary design should take fire safety into account for a strong and secure design. The strength and stiffness properties of steel

and concrete, which are the building blocks of reinforced concrete beams, steadily deteriorate as the temperature rises. In order to analyze the load-bearing capacity of reinforced concrete beams following a fire, one must comprehend the cross-sectional temperature distribution. The material's thermal characteristics, such as heat capacity and thermal conductivity, govern this.

In the event of a fire, the building will be subjected to temperatures as high as 1200°C. Notable damage will inevitably result in the concrete surface deteriorating due to extraction. Although the phenomenon's symptom was initially noticed a long time ago, its calculation is still unclear. This is because spalling is a complex phenomenon that can be influenced by a number of things, beginning with concrete resistance, moisture level, density, fire intensity, lateral frame, cargo state, kind of aggregate, rate of heating, sample dimensions and shape, and so on. When these elements come together, concrete may collapse in certain ways close to the fire-exposed face [12].

Calculating the fire load density in the compartment is a relatively new technique used by fire engineers to assess fire exposure. Then, ascertain the temperature of the compartment at various times based on the ventilation parameters and the suspected combustion source. The analysis also takes into account other factors, such as the effect of active fire prevention systems. Sprinklers and fire departments, for instance, can help contain a fire. Changes in the fuel load over time and variations in the ventilation conditions during the fire can have an impact on the size and rate of fire growth, as measured by fire analysis. This kind of fire study is limited to very large or uncommon buildings and requires specialized software and substantial training. The strength and modulus of elasticity of steel bars decrease with increasing temperature; however, the amount of decline in strength and modulus is dependent upon the rate of increase in fire temperature as well as the insulating qualities of the concrete [28].

The majority of the variables that affect a building's safety during a fire are essentially unpredictable. No building system can be planned and constructed to be completely free of fire threats when there is uncertainty. Rather, occupant education, building systems engineering, structural and construction engineering, and other strategies should be used to manage the danger of fire. The mechanical characteristics of the concrete are altering during the fire. The concrete is unable to return to its former state when cooling. In order to further advance our knowledge of this issue, this research offers an assessment approach

based on suitable analysis techniques. This method will be used on the model of steel bars, concrete, and reinforced concrete beams subjected to fire.

The analysis approach simulates RC beams utilizing coupled displacements using ABAQUS software to examine the behavior of fire-reinforced concrete beams. The study makes use of four parameters: temperature duration, temperature level, compressive strength, and concrete cover. The ABAQUS model is used to simulate the behavior of beams exposed to fire in order to accomplish these goals. Conduct an analysis to determine how these various parameters affect the beam's strength.

1.2 Assumption

The temperature of the surface of the heating limit of the RC part is supposed to be the same as the combustion temperature of the ISO gas; that is, it assumes that there is no loss of heat from convection at the interface between the fire and the heated RC surface. The structural member or assembly's measured fire resistance is determined by the furnace's specific properties, geometry, loading force, and exposure to fire. In the laboratory, the fire resistance survey of the materials is carried out by subjecting the element to fire and observing its behavior. Based on these fire tests, the numerical and analytical procedures were created as a cost-effective replacement for laboratory testing.

The purpose of this work is to advance the field of structural engineering's comprehension of the ideas underlying fire safety structural design. Establishing a reinforced concrete beam model with various parameters during a fire is the primary goal of the research provided by the current study in order to discover sensitive parameters. Fires in reinforced concrete structures The sensitivity of the parameters that occur during the combustion of reinforced concrete beams has been studied by various researchers up to this point in the study. The first goal of this research project is to examine how RC beams behave structurally when they are in a fire. Finding the reinforced concrete beam's ultimate bending moment that remains after fire damage is another goal.

1.3 Statement of the Problem

In our country, Ethiopian data on fire accidents now a day significantly increases due to undefined sources, especially ADDIS ABABA city reports. The damage designer must know the parameters that minimized the damage throughout the accident. This study

mainly focuses on the sensitive parameter that determines the influence of fire on reinforced concrete beams while considering parameters on the concrete cover, concrete compressive strength, fire duration, and level of fire to determine performance. The most important part of this study covers up to a temperature level of 700 °C. In the other study, it was difficult to do experimentally up to 700 °C.

Finally, in my country's Ethiopian Building Code, there is no detailed and sufficient design for the way reinforced concrete structures fare in fire.

1.4 Objective

1.4.1 General Objective

This study's primary goal was to determine how sensitive certain parameters were to the way reinforced concrete beams behaved when mechanical loads were applied. Assume that the temperature only changes within the depth of the beam or frame beam and remains constant throughout the member. This can be assumed to be a one-dimensional problem, so the stress obtained due to temperature changes is the normal stress. The value of researching reinforced concrete beams' fire performance aids in the designer's consideration of fire's impact. Parameters used for the study, like the compressive strength of the concrete cover, temperature change, and duration of temperature.

1.4.2 Specific Objective

- Improve structural engineers' understanding of the design concept of fire conditions.
- To understand which parameter is more sensitive.
- Examine how reinforced concrete beams behave structurally during a fire.
- After a fire damages reinforced concrete beams, find the remaining ultimate bending moment.
- Study how fire affects reinforced concrete beams by considering concrete cover, temperature, duration, and compressive strength.
- The effect of fire on axial fire behavior and estimate the percentage of bending loss and relative loss of compressive strength.
- Understand the characteristics of RC beams by changing one parameter while the other parameter does not change and changing two parameters for another constant.

- Enter the fire safety design at what temperature and time level the reinforced concrete beam can lose its strength.

1.5 Thesis Hypothesis

When this study is through

- We understand what level of fire occurs, and fire protection measures for reinforced concrete beams are essential.
- We can establish an effective fire protection mechanism in our design steps.
- How does the reinforced concrete beam behave in terms of different levels of fire exposure parameters?
- We know which parameter in the reinforced concrete beam is more sensitive than another.
- We understand the failure modes of exposed steel beams after a fire.

1.6 Research Application

In our country, Ethiopia, fire accidents increase at a maximum rate due to this phenomenon. There will be further studies on fire. Such fulfillment articles yield significant discoveries and research results of great value, as past practices have demonstrated, and the outcomes of this study will be beneficial.

- Used for reinforced concrete beams designed for fire protection.
- Understanding the effect of fire up to 700 °C
- Assessing reinforced concrete beams subsequent to the fire incident
- By organizations that work to make buildings fire-safe.
- The construction industry focuses on the area of study.
- To give input for the code regarding the fire.

1.7 Limitations

In this study, some limitations come from the level of study and resource capacity. The following are the limitations:

- The influence of moisture content was overlooked.
- Following cooling, the loading test is conducted.
- No test of simultaneous burning and loading was taken into account.

- Only concentrated loads on the beam's midspan are taken into account.
- Only one sample dimension was taken into account; the impact of the other dimensions was not.
- Just basic support requirements are taken into account.

1.8 The thesis's contents

There are five chapters in this work. The first chapter serves as an introduction, covering the background research on the behavior of fire-reinforced concrete, the problem statement, the research objectives, and the application of the findings with some limitations to the present construction process.

The majority of the second chapter is devoted to reviewing and presenting works of literature that address the effects of reinforced concrete and fire. It covers factors that affect reinforced concrete's fire resistance, factors that are impacted by fire exposure, fire test procedures, fire endurance acceptance standards, and suggestions for the best reinforced concrete fire resistance.

The utilization of materials, the analysis methodology, and the finite element model for the sensitivity analysis of fire-exposed reinforced concrete beams are all covered in detail in the third chapter.

The final chapter contains the conclusion and recommendations of the finite element model on the sensitivity analysis of reinforced concrete beams exposed to fire. The fourth chapter deals with the results and discussion of compressive strength, concrete cover, fire intensity, and duration on reinforced concrete beams discussed in this chapter.

CHAPTER 2 LITERATURE REVIEW

2.1 Property reinforced concrete in fire

In general, concrete resists fire well. It has a longer flame-resistance time without weakening. The coarse aggregate used will determine how fire affects the characteristics of the concrete. Concrete aggregate is divided into three types: carbonate, siliceous, and lightweight. [13]

There are intricate relationships between concrete and fires because of the composition of concrete and the high temperatures that are frequently present in fires. Concrete is by no means a uniform substance. Aggregates, standard steel (or other) rebar, and gel cement compounds make up its composition. It is difficult to define or model the behavior of the composite system during a fire because each of these components reacts differently to heat exposure.[5] This section discusses the mechanical properties of concrete that vary with temperature. These properties include compressive and tensile strength, modulus of elasticity, shear modulus, and coefficient of thermal expansion.

Concrete will change in certain ways when it comes into contact with fire. Below is a discussion of a few of the properties or relationships that will be impacted. The kind of aggregate (among the three) that is employed greatly influences affection.

- Compressive strength
- Thermal expansion
- Modulus of elasticity

Compressive strength is one of concrete's best qualities, and a specimen that is comparable to a typical concrete sample can be used to study how fire affects it. Compressive test specimens with varying aggregate compositions will be heated to the desired temperature at intervals of either 50°C or 300°C per minute until the interior temperature of the concrete reaches a stable state. Either of the two criteria can be used to describe steady-state furnace burning.

- When test samples are heated or burned, internally fixed thermocouples record variations in internal temperature within 5 °C.
- The interior temperature does not deviate from the intended temperature by more than +/- 5 °C [11].

2.2 Concrete's Thermophysical Properties

At least the following material parameters are necessary for unstable conduction heat transfer analysis:

- Thermal conductivity, λ (W/m.K)
- Specific heat, C_p (J/kg.K)
- Density, ρ (kg/m³)

The thermal characteristics of concrete that have been recorded come from small-scale experiments. It is expected that full-scale members will behave differently, particularly in terms of moisture transport. Consequently, it should be assumed that these characteristics are only helpful in estimating the temperature response of a structural concrete member.

2.2.1 Conductivity Concrete with Thermal Conductivity

The concrete's thermal conductivity (λ) is mainly determined by the type of aggregate and density of the concrete. ACI 216.1 [17] lists values of λ for concrete with densities between 800 and 2,400 kg/m³. The mean values of the following for the thermal conductivity of NSC with normal weight and lightweight aggregate may be utilized for basic analyses:

Normal weight aggregate NSC: $\lambda = 1.3$ W/m.K.

Lightweight aggregate NSC: $\lambda = 0.5$ W/m.K.

When temperature-dependent thermal conductivity is required, a more thorough examination is required. According to EN 1992 1-2 [15] (Figs. 2.1 and 2.2), concrete's thermal conductivity and specific heat capacity are defined; its density is assumed to be constant at 2300 kg/m³. The impact of moisture in concrete is implicitly taken into account by adding the latent heat of the evaporation component to the concrete's specific heat capacity. This latent heat, represented by the symbol CC , peaks between 100 and 115 degrees Celsius and falls linearly between 115 and 200 degrees Celsius. According to Fig. 2.1, at 1.5% moisture content, the C_c peak is equal to 1470 J/(kg. °C) and 2020 J/(kg. °C), respectively.

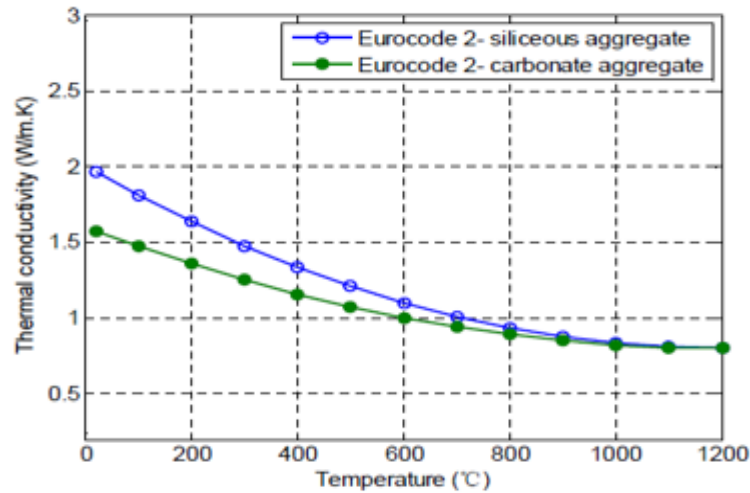


Figure 2. 1: Thermal conductivity concrete

2.2.2 Concrete with a Volumetric Specific Heat

Concrete's volumetric specific heat, or $\rho.Cp$, can be calculated using the following values for basic analyses (EC2 2002):

Concrete with a normal weight: $\rho.Cp = 2.6 \text{ MJ/m}^3\text{K}$

Lightweight concrete: $\rho.Cp = 1.5 \text{ MJ/m}^3.\text{K}$

The normal-weight aggregate concretes have a higher volumetric specific heat than the lightweight aggregate concretes due to differences in their respective volumetric specific heats. The temperature rise inside the concrete part and on its unexposed surface will be impacted by this, just like it is by thermal conductivity.

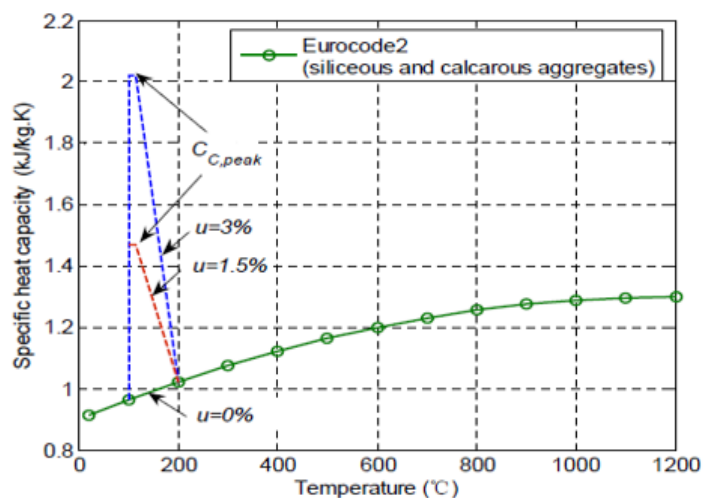


Figure 2. 2 Specific heat capacity concrete at high temperature

2.2.3 Concrete with thermal Expansion

The expansion of concrete's dimensions brought on by heating is known as thermal expansion. Experiments were conducted on three distinct aggregate types to examine the expansion of concrete when exposed to high temperatures; the findings are displayed in Fig. 2.3 [17].

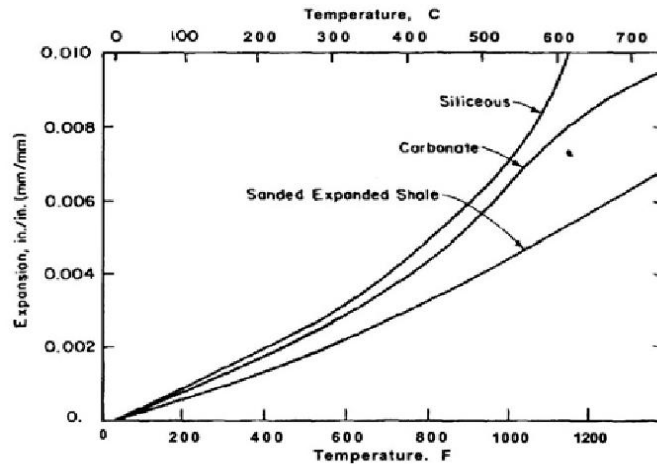


Figure 2. 3- Concrete's thermal Expansion at High Temperatures [17]

2.2.4 The explicit modulus of elasticity

Using Fig. 2.4, the modulus of elasticity of concrete may be defined in three different ways: the tangent modulus of elasticity at a given position on the stress-strain curve is equal to the slope of a line that is tangent to it.

The initial tangent modulus of elasticity is represented by the slope of the curve at the origin. The slope of the line passing through the origin and the point on the curve that represents the stress is the secant modulus of elasticity at that particular stress. The point corresponding to 0.4 f'_c is often used to define the secant modulus of elasticity [17].

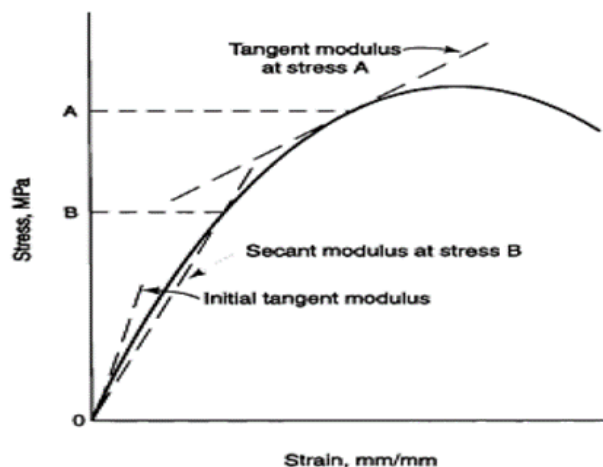


Figure 2. 4- The modulus of elasticity is defined by the stress-strain curve [17].

2.3 A fire's Effect on Reinforced Concrete Beams

2.3.1 RC Beam Transient Thermal Analysis

Thermal analysis and mechanical analysis are the two phases in the thermomechanical study of RC beams. The first step's temperature distribution will serve as the foundation for the mechanical analysis that comes next. In order to streamline the thermal study, Figures 2.5 and 2.6 depict the yield strength and Young's modulus of steel at elevated temperatures, respectively, based on air temperature. Eurocode defines the mechanical parameters of steel bars [15].

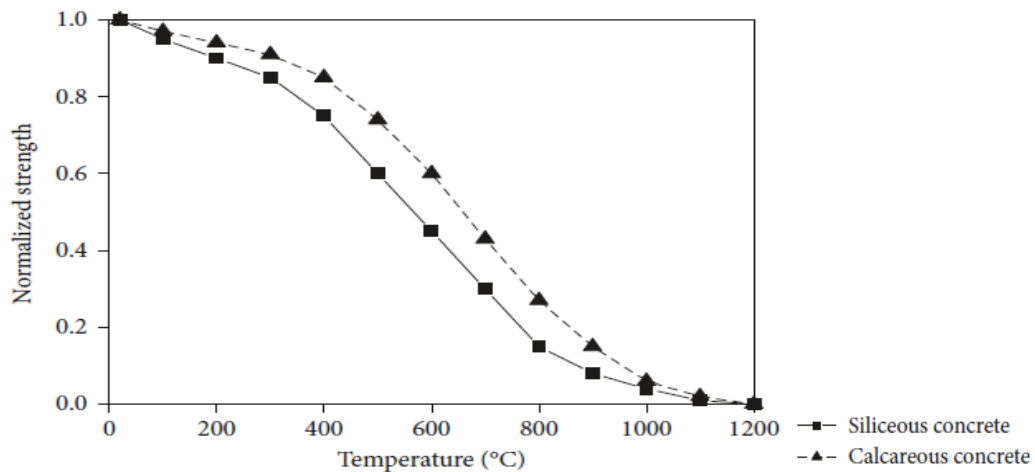


Figure 2. 5- The decrease of steel's mechanical properties at elevated temperatures.

Highly temperature-dependent material properties must be accurately specified in numerical simulations because of their impact on the mechanical and thermal response of RC beams subjected to fire. The link outlined in the Eurocode will be put into practice by this study [15]. Generally speaking, it has minimal impact on the thermal response. As a result, a constant value of 2300 kg/m³ is considered in this investigation. The concrete's specific heat (c) and conductivity (k) can be represented as functions of temperature, according to the Eurocode [15].

$$k = 1.6 - 0.16 \frac{T}{120} + 0.008 \left(\frac{T}{120} \right)^2 \dots\dots\dots(1)$$

$$C = 900 + 80 \frac{T}{120} - 4 \left(\frac{T}{120} \right)^2 \dots\dots\dots(2)$$

2.4 Steel Reinforcement

Rebar's thermal performance is not taken into account and therefore has little impact on the heat conduction analysis. Several experimental studies and design guidelines have regulated the changes in the mechanical characteristics of steel bars and concrete. A constitutive model used to explain the uniaxial compression of concrete at the evaluation temperature is the Eurocode Model [15]. Since these crucial characteristics govern the stress-strain relationship of concrete at a specific temperature, the compressive strength f_{ck} and related deformation ϵ_{c1} are represented as a function of temperature. Elevated temperatures, as illustrated in Fig. 2.6, will surely cause concrete's compressive strength to drop and its corresponding deformation to increase, as indicated in [15]. The modulus of compressive concrete is the linear connection between stress and strain.

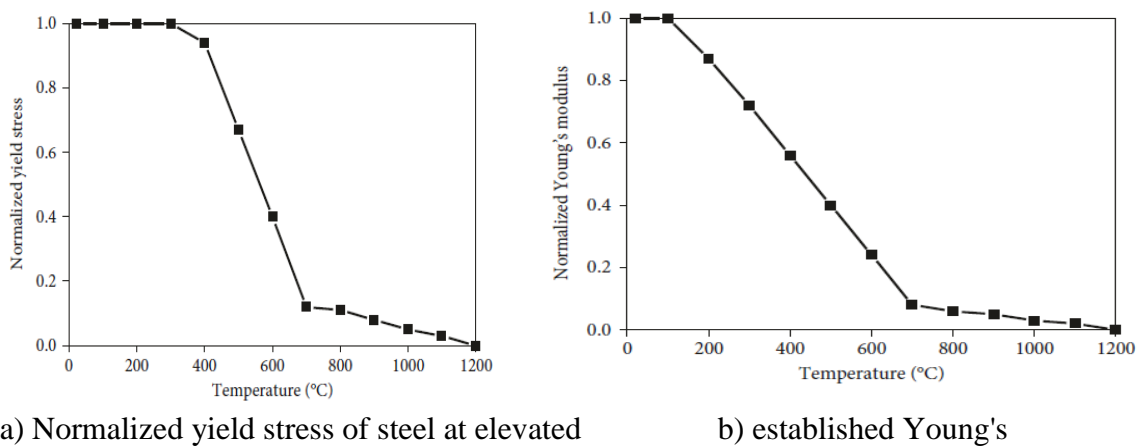


Figure 2. 6 Young's modulus and normalized yield stress of steel at high temperature

The intricacy of the temperature distribution is avoided, and the steel is insulated by simply designating a specific depth of concrete cover for the steel bars in the composite structure. In order to replace the uncertainties in the specifics of the thermal reaction, a great deal of testing is done, primarily using standard heating curves. The results are provided as a "fire resistance" time for various types of concrete, which is usually a function of thickness or cover [7].

After cooling, some of them are reversible, but others are irreversible and can seriously impair the concrete structure following a fire. There is some liquid water present in most porous concrete. It will start to evaporate at temperatures above 100°C, which typically causes pressure to build up in the concrete. In actuality, the boiling point temperature range tends to stretch from 100°C to roughly 140°C because of the effect of pressure. When the temperature rises over the humidity plateau to roughly 400°C, the calcium hydroxide in

the cement will start to dehydrate, producing additional water vapor and drastically decreasing the material's physical strength.

Additional alterations in the aggregate may transpire at elevated temperatures. For instance, mineral transformation causes quartz-based aggregates to expand in volume at about 575 °C, but limestone aggregates start to break down at about 800 °C. When considered separately, the aggregate's thermal reaction could be simple, but the aggregate's alterations can have a significant impact on the concrete's overall response. For instance, spalling and cracking may result from the aggregate and cement matrix expanding differently.

The aggregate of these chemical and physical alterations in concrete will result in a decrease in the material's compressive strength. In actuality, the approximate values for the essential temperatures for a considerable drop in strength are as follows: 650°C for light-weight sand concrete, 660°C for carbonate, and 430°C for silica. When temperatures are lower, the impact of temperature on strength can also vary greatly, according to external conditions and composition, such as how much moisture has "sealed" the concrete [5]. But as was already indicated, because of the severe temperature gradients that are typically present inside the depth of the material, all of these temperature connections only offer an indirect connection to the fire resistance of concrete structures.

Only when any steel reinforcement loses its effective strength as a result of heating can structural failure usually occur. Significant research has been conducted for many years on the specifics of the chemical and physical changes that occur in concrete at high temperatures. Unfortunately, most of these tests are conducted on certain pre-established heating systems, which might not accurately simulate the heating conditions found in actual fires.

- Use alternative temperature-time relationships that are only appropriate for particular applications.
- Slow heating lowers the internal temperature gradient.
- The temperature-time curve is used in the typical fire test.

As a result, the impacts of both chemical and physical alterations connected to the typical thermal gradient of a fire have rarely been assessed together. Thus, the systematic variation of the material surface and depth's heat exposure and the interpretation of the potential conditions of these studies in real fires that establish the "worst-case" scenario represent a significant research topic that remains unresolved.

Concrete constructions' altered characteristics cannot be undone after a fire. Steel constructions are not like this. Cooling typically returns steel buildings' constituent materials to their original state. This is because the chemical and physical characteristics of the cement itself have irreversibly changed. These alterations can serve as a guide for determining the maximum exposure temperature, based on an examination of the concrete's surface state following the fire. [1, 12]

Tensile or bending strength failure will happen to load-bearing steel slabs if the steel loses strength as a result of heat. When L is the span, this mechanism is typically identified by the deflection in the centre of the $L/30$ span. The steel parts may also break and lead to the associated tensile failure of the concrete when the bond between them is broken. Although it is not well defined empirically, the tensile strength of concrete also affects torsion or shear failure. Lastly, the reduction of concrete's compressive strength in the temperature-dependent compression zone is typically linked to compression failure. In actuality, a large number of these requirements are tied to the structural performance of the individual components in place that is, to the constraints and reinforcement that other structural elements offer and so cannot be taken into account separately. In actual fires, concrete structures can collapse for a variety of causes. The inadequate craftsmanship and steel bar continuity are the cause. Realistically speaking, the floor slab's thermal expansion has generated a significant horizontal displacement that the structure is unable to withstand or adjust to.

2.5 The Reinforced Beam's Mode of Failure in Fire

2.5.2 Spalling

The strength of the reinforced concrete structure will be negatively impacted by spalling as the heating of the steel bars improves. The concrete covering on the steel bars will be greatly reduced or removed by spalling, exposing the steel bars to high temperatures and reducing their strength, which will result in poor overall structural and mechanical qualities.

By decreasing the cross-section of the concrete that can support the given load, spalling also significantly affects the structure's physical strength by increasing stress in the remaining concrete. This is significant because, before other detrimental effects of heating on the strength of concrete, spalling can manifest at relatively low temperatures.

The mechanism responsible for spalling is widely thought to include high thermal stress brought on by sudden heating and/or significant pressure building in porous concrete, where the water evaporation prevents the concrete structure from dissipating. These processes cause material blocks to be ejected from the surface layer and fissures to form. More precisely, it has been established that a high temperature gradient within the material and a moisture content of at least 2% are the primary requirements for spalling. Regarding the latter, a value of around 5K/mm is the lowest, and debris of 78K/mm is expected to occur [15].

2.5.3 Cracking

In general, the processes that result in flaking and cracking are thought to be identical. Rather than producing explosive flakes, thermal expansion and dewatering of the concrete as a result of heating might cause fissures in the material. These fissures may allow the steel bar to heat up directly, increasing the risk of heat stress and cracking. There are situations where fissures might act as pathways for a fire to spread from one compartment to another. [6]

A concrete building that had been subjected to fire was the subject of a case study on cracking, with a focus on the depth to which the fracture penetrated the concrete. Research has shown that the temperature of the fire affects the penetration depth and that cracks typically reach the depths of concrete components. Although the majority of the damage was contained in the area near the fire's source, the concrete's discoloration and cracking revealed that the temperature of the concrete around the steel bars had reached 700°C.

Cracks that penetrate more than 30 mm are a result of the brief heating-cooling cycle that occurs after the fire is put out. The significance of stress conditions in concrete needs to be taken into consideration. Thermal expansion can produce a compressive load that is highly helpful in compacting the material and preventing the formation of cracks [15]. As a result, the specimen with a reduced load experiences a considerably smaller reduction in compressive strength and elastic modulus.

2.6 Reinforcement's Reaction in a Fire

Steel behaves differently in a fire than concrete does, and it is possible to forecast with some degree of accuracy how strong steel will be at a particular temperature. In general, some individuals believe that temperatures exceeding 250–300°C should not be allowed to affect steel bars. This is due to the well-known "blue brittleness" that low-carbon steel

displays between 200 and 300°C. Steel and concrete both show comparable thermal expansion up to 400°C; however, steel will expand more at greater temperatures than concrete. The load-bearing capacity of steel bars will drop to about 20% of their design value if the temperature hits roughly 700°C.

Through the creation of impermeable areas that can hold water, reinforcing bars can also have a major impact on the transmission of water within heated concrete elements. Water begins to circulate around the steel bars as a result, raising the pore pressure in some concrete and raising the possibility of spalling. However, these water-trapped places will also alter the heat transfer around the steel bars, lowering the interior concrete's temperature [4].

2.7 Factors Influencing Reinforced Concrete's Fire Performance

The performance of reinforced concrete structures in fire is influenced by numerous elements. These variables differ from component to component, meaning that one variable that influences the performance of slabs made of reinforced concrete cannot also influence the performance of columns made of reinforced concrete. Thus, the criteria (factors) influencing the behavior of reinforced concrete slabs under fire will be covered in this section. While reinforced concrete slabs are the focus of this analysis, other reinforced concrete components are also impacted by similar parameters. The top surface temperature rise of concrete slabs is primarily determined by the slab's thickness, unit weight, moisture content, and kind of aggregate. [17,20] Air content, aggregate moisture content during mixing, maximum aggregate particle size, water-cement ratio, cement concentration, and slump are additional variables that influence temperature rise but have a smaller effect.

2.7.2 Concrete unit weight's impact on fire endurance

In general, fire endurance rises as unit weight falls [17]. The impact of unit weight in structural concrete may be overshadowed by the type of aggregate. Figure 2.7 illustrates the relationship between unit weight (oven-dry) and fire endurance for low-density concretes.

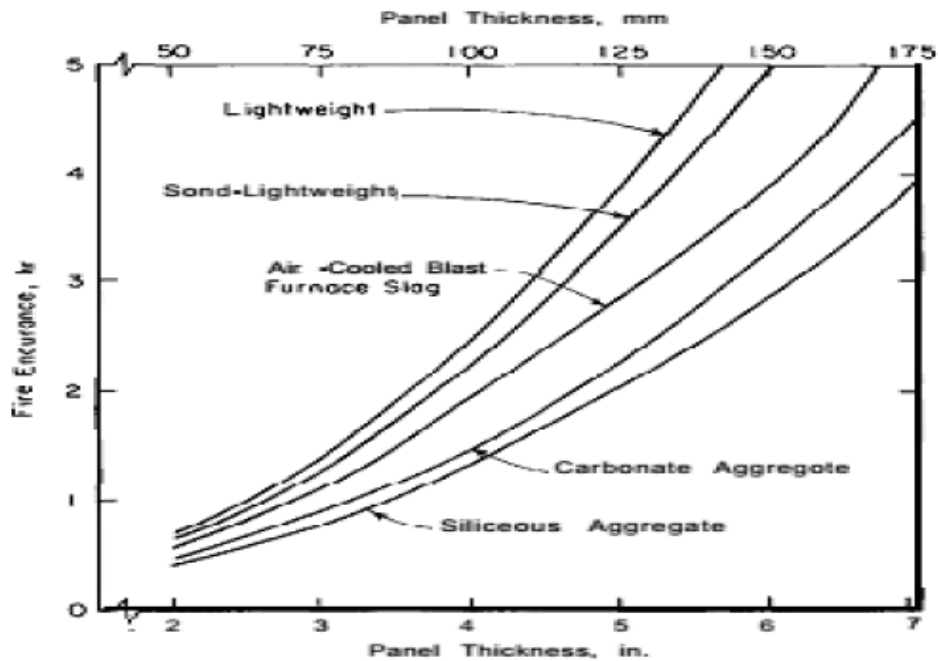


Figure 2.7- Concrete slab fire resistance as a function of slab thickness and aggregate type.
 [Based on the unexposed surface experiencing a temperature rise of 250 F, or 139 0C] 17)

2.7.3 Moisture conditions' Impact on the fire Resistance of concrete

The concrete's moisture content and drying method have an impact on the material's fire resistance during testing. In general, fire resistance will decrease with decreasing moisture content or high-temperature drying at 120 to 200 °F (49 to 94 °C). Concrete slabs' fire resistance can be modified based on the humidity content and dry atmosphere [18].

2.8 Reinforced Concrete Fire Test

Establishing the fire resistance grade or fire resistance duration that we want the structural component to have is the first step in building a reinforced concrete structural component for a fire of the necessary level (fire resistance duration). The next stage of this procedure is to figure out how to make structural elements fire-resistant in the allotted amount of time. This can be accomplished by adjusting the variables that impact the structural elements' fire resistance. As was mentioned in Section 2.3, some parameters have a direct relationship with fire resistance, meaning that increasing one will increase fire resistance; however, other parameters have an inverse relationship with fire resistance, meaning that increasing one will decrease fire resistance.

As this section has explained, fire testing structural components is the most effective way to investigate the relationship between fire resistance and its affecting characteristics, both direct and inverse. In ASTM [18], comprehensive protocols, failure criteria, and several

kinds of fire resistance tests for various structural components are satisfactorily covered. The fundamental idea behind the fire resistance test is to put the sample in a typical fire and time it to achieve a certain temperature. After exposure to fire, use the designated conventional fire hose blast as needed. Under these fire exposure settings, this test offers a relative estimate of comparable components' fire resistance test reaction. The display does not depict every possible fire scenario since each one is unique in terms of the quantity, kind, and distribution of the fire load as well as ventilation, compartment size, and arrangement. The results of the fire resistance test will also be impacted by modifications made to the sample's size, composition, and assembly technique, among other test parameters. Changes need to be evaluated during on-site construction for these reasons [18].

Test specimen: under the constraints set by the testing facilities, attempt to replicate the real building as nearly as feasible. Before testing, all specimens must be conditioned to reach a moisture content that is similar to what is seen in the field.

A standard fire is one that is managed to maintain a set temperature for a predetermined amount of time. Despite not being typical of structure-related fire incidents, it is acknowledged as the norm.

Time: Temperature curve of standard fire—a curve showing an increase in the temperature of fire with time. Fig. 2.8 shows the variation of temperature with the time of the standard fire.

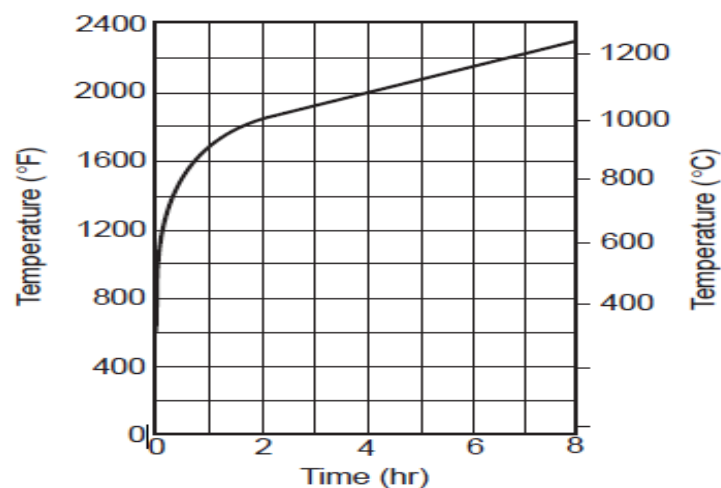


Figure 2. 8- Time-Temperature Curve [17]

Test furnaces are locations where materials are assembled or arranged to test them for fire. This location features a system of regulated fuel flow for burning as well as temperature sensors (thermocouples) to measure the controlled temperature.

2.8.2 Roofs and floors made of reinforced concrete Test of fire

This reference to "the roof" refers to the reinforced concrete roof. This test process necessitates fire exposure at the bottom of the test specimen and is applicable to floor and ceiling components with or without connections, leather, or suspended ceilings. It is necessary to infer the two classes of fire resistance from the testing of free and thermally expanding components [18].

2.8.3 Size of Specimen

The fire-exposed area must be at least 180 square feet (16 square meters), with no dimension smaller than 12 feet (3.7 meters). If they are a component of the building being tested, structural elements must be located inside the combustion chamber with a minimum of 8 inches (203 mm) of side clearance from the walls. In the furnace, specimens that depict architectural designs where thermal expansion is restricted must be restricted in this way [18].

2.8.1 Reinforced concrete restrained beam Fire test

The fire resistance test of reinforced concrete beams is the same as the fire resistance test of the reinforced concrete floor (slab). The fire test of reinforced concrete slabs supported by beams is introduced. The only difference is that the beam will only be considered and tested as a whole. When used with a floor or ceiling structure, the resulting fire resistance rating should apply to beams that have a heat dissipation capacity equal to or greater than that of the floor or ceiling used in the test. The fire-resistance rating established by this method does not apply to beam sizes smaller than the test size [18].

2.8.2 Specimen Dimensions and Features

The members must be tested in a horizontal position, and the length of the beam exposed to the fire cannot be less than 3.7 meters. The width of the representative floor or ceiling structural elements used in sample testing should not be greater than seven feet (2.1 meters), and they should be positioned symmetrically with respect to the beam. A constrained beam resists longitudinal thermal expansion in a way that is similar to the limitations in the structure that is shown, including a portion of a floor or ceiling element that makes up a whole beam (like a composite steel or concrete structure). With the

exception of the beam portion that is part of the design, it does not support or restrict the perimeter of the sample floor or ceiling elements [18].

2.9 Reinforced concrete's minimum thickness and concrete cover for fire

The performance of reinforced concrete during a fire accident is determined by a number of parameters (dimensions). A thorough understanding of these parameters and their effects on the fire endurance behavior of reinforced concrete will aid in the design process of making reinforced concrete fire resistant for the desired duration. Some of the parameters are costly or challenging to improve for better fire endurance; for instance, it is challenging to vary the modulus of elasticity of reinforced concrete behavior for better fire endurance. It is simpler to change parameters for a higher fire endurance rating, such as the thickness of a member and the concrete cover over the reinforcing steel bar. The test findings indicate that the type of aggregate used will affect the fire resistance of concrete structures [17, 20]. The minimum thickness specifications for cast-in-place floor and wall slabs of various concrete kinds and fire resistance classes are compiled in Table 2.1. The minimum column diameters for various concrete materials and fire resistance levels are compiled in Table 2.2 [20]. The minimum thickness of the concrete cover for the reinforcement is another issue to take into account while adhering to fire-resistive requirements [17, 20].

Table 2. 1 - Minimum thickness required for roof and floor slabs made in place, inch (mm) [20].

Concrete type	Fire resistance rating				
	1 hr.	1.5 hr.	2 hr.	3 hr.	4 hr.
Siliceous aggregate	3.5 (88.85)	4.3 (109.15)	5 (127)	6.2(157.40)	7.0(177.70)
Carbonate aggregate	3.2 (81.25)	4 (101.55)	4.6 (116.80)	5.7(144.70)	6.6(167.60)
Sand-lightweight	2.7 (68.55)	3.3 (83.80)	3.8 (96.50)	4.6(116.80)	5.4(137.10)
Lightweight	2.5 (63.45)	3.1 (78.80)	3.6 (91.40)	4.4(111.70)	5.1(129.50)

Table 2. 2- Minimum dimensions of a concrete column in inches (mm) [20].

Concrete type	Fire resistance rating				
	1 hr.	1.5 hr.	2 hr.	3 hr.	4 hr.
Siliceous aggregate	8(203.10)	9(228.45)	10(253.85)	12(304.60)	14(355.35)
Carbonate aggregate	8(203.10)	9(228.45)	10(253.85)	11(279.20)	12(304.60)
Sand-lightweight	8(203.10)	8.5(215.80)	9(228.45)	10.5(266.50)	12(304.60)

CHAPTER 3 MATERIAL AND METHODS OF ANALYSIS

3.1 Sample Group for Finite Element Model

3.1.1 For Cube concrete compressive strength F_{CK} -25Mpa and 30Mpa

In this study, 150mm cube compressive strength was used. All samples and six groups of cube compressive strength were prepared for finite element analysis. Each group contains seven individual samples that were exposed to different intensities of the fire. In addition to this, to understand the effect of parameters such as concrete compressive strength, duration of the fire, and intensity of the fire, one sample is kept as a reference without exposure to a fire load and is listed below the table..

Table 3. 1 For concrete cube compressive strength F_{CK} -25Mpa.

Group	Duration (hrs.)	Concrete Compressive Strength(Mpa)	Temperature (°C) within interval 100
Group 1*	0	25	20
Group 1	2	25	100-700
Group 2	3	25	
Group 3	4	25	

Table 3. 2 For concrete cube compressive strength F_{CK} -30Mpa.

Group	Duration (hrs.)	Concrete Compressive Strength(Mpa)	Temperature (°C) within interval 100
Group 1*	0	30	20
Group 1	2	30	100-700
Group 2	3	30	
Group 3	4	30	

3.1.2 Reinforced concrete beam dimension for the model used

For reinforced concrete beams, the finite element model uses the same cross-section for all groups of samples, and the beam is also simply supported with a cross-section of growth depth (D) 250mm, width (B) 150 mm, and length (L) 1100 mm in dimensions.

For the finite analysis model, use 15mm and 25mm concrete covers as per the Eurocode standard, 25MPa and 30MPa cube compressive strengths, a duration of 2 hours, 3 hours, and 4 hours, and a fire intensity of 100oC–700oC within an interval of 100°C.

For a reinforced concrete beam with a concrete cover of 15mm and 25 mm, sample groups were grouped using the parameters to create a total of twelve beam samples for the model that were exposed to a different level of fire load, and also one reference beam was present for the analysis to determine the effect of fire load.

Table 3. 3 Sample group of concrete cover 15mm beam with 25Mpa compressive strength

Group	Concrete Cover (mm)	Concrete Compressive Strength(Mpa)	Duration (hrs.)	Temperature (°C) within interval 100
Group 1*	15	25	0.00	20
Group 1	15	25	2	100-700
Group 2	15	25	3	
Group 3	15	25	4	

Table 3. 4 Sample group of concrete cover 25mm beam with 25Mpa compressive strength.

Group	Concrete Cover (mm)	Concrete Compressive Strength(Mpa)	Duration (hrs.)	Temperature (°C)
Group 1*	25	25	0.00	20
Group 1	25	25	2	100-700
Group 2	25	25	3	
Group 3	25	25	4	

Table 3. 5 Sample group of concrete cover 15mm beam with 30Mpa compressive strength

Group	Concrete Cover (mm)	Concrete Compressive Strength(Mpa)	Duration (hrs.)	Temperature (°C)
Group 1*	15	30	0.00	20
Group 1	15	30	2	100-700
Group 2	15	30	3	
Group 3	15	30	4	

Table 3. 6 Sample group of concrete cover 25mm beam with 30Mpa compressive strength.

Group	Concrete Cover (mm)	Concrete Compressive Strength(Mpa)	Duration (hrs.)	Temperature (°C)
Group 1*	25	30	0.00	20
Group 1	25	30	2	100-700
Group 2	25	30	3	
Group 3	25	30	4	

3.2 Introduction to Abaqus

For those who will be utilizing ABAQUS for research and would like to have a basic understanding of its primary capabilities to get started efficiently, the "ABAQUS/CAE Introductory Training Course" is appropriate. A general-purpose nonlinear finite element analysis program, ABAQUS may offer many solutions for engineering applications in the fields of mechanical, structural, civil, biomedical, and related fields. includes boundary conditions, meshing, material attributes, geometric modeling, and other topics to produce a comprehensive project prognosis.

All-in-one ABAQUS environment ABAQUS/CAE offers a straightforward and uniform interface for ABAQUS/Standard and ABAQUS/Explicit simulation creation, sending, monitoring, and evaluation. It's a user-friendly environment for novices as well.

3.3 Analysis Methodology

To understand the response of the structure to thermal loads, each stage of the analysis must be considered separately. General procedures for transient thermal stress analysis of RC structures at Abaqus include:

- a) Build a two-dimensional or three-dimensional model of the structure. The model combines geometry (concrete and steel bars as steel bars), suitable material properties, and boundary conditions.
- b) Apply a thermal load to the required surface of the structure created by the transient fire (in the form of a transient curve of temperature versus time).
- c) Based on the "standard time-temperature" curve applied in ISO 834 or ASTM E 119 (as shown in Figure 3.1), The rest of the structure is believed to be exposed to room temperature.

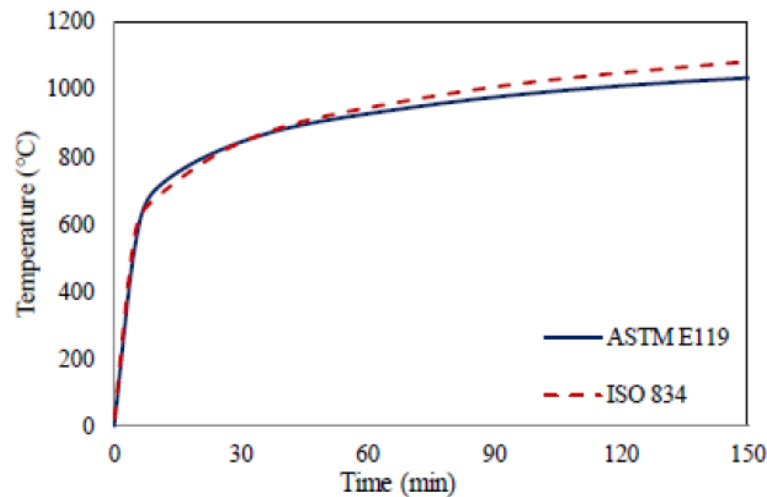


Figure 3. 1 Standard Fire Curve according to ASTM E119 and ISO 834.

- d) Apply a mechanical load (concentric load at the center of the beam) on a suitable surface to simulate static gravity and live gravity service loads during fire exposure. Furthermore, at each time point obtained in step 2, the thermal load is applied to the finite element model of the structure in the form of node temperature, and the deflection and deformation of the structure are calculated.
- e) Evaluate the total deflection, thermal, mechanical, and total deformation of the finite element structure model at different points of the cross-section of the half-span beam during fire exposure. The deformation field of the first loading step in the model (due to the applied gravity load) is used to verify the correct behavior of the model and the correct modeling of constraints and loading conditions.

FEA is used to understand the response of existing structural elements in RC buildings within the critical load time. Here, it is carried out with reinforced concrete beams, which have a variable grid concrete cover, variable compressive strength, variable fire resistance, and variable fire duration. The finite element software used is ABAQUS modeling analysis, using nonlinear static analysis. The problem considered in the research is to conduct an experimental analysis to study the resistance of each component and the concrete under various parameters. This method provides the true behavior of the structure, which can be easily achieved through finite element analysis without spending less time. This analysis can accurately predict the response of components affected by various parameters.

3.3.1 Contact

The structure being tested contains a large number of physical contacts. Surface-to-surface contact is provided by ABAQUS in typical static analysis. You have to be quite careful while mapping the primary and secondary surfaces in ABAQUS when applying face-to-face contact. The primary surface of the main body ought to be composed of a solid substance, in theory.

3.3.2 Model geometry, Mesh size, and Element type

The ABAQUS element library offers several hexahedral, roof, and beam elements with various properties for comprehensive modeling. The 3D finite element is the best option for comprehensive modeling of joint behavior, according to numerous in-depth finite element evaluations. First-order units might be the best option for non-linear issues because of their high precision and low processing resource requirements. The Eight Head Reduction Integrated Brick Element (C3D8R) was employed in this investigation. This unit has the advantage of correct integral modeling of the constitutive law and the avoidance of the shear lock issue. In connection areas with high stresses and strain gradients, fine meshes are necessary.

3.4 MODELLING OF CONCRETE IN ABAQUS

The material qualities of steel bars and concrete have a significant impact on the mechanical and thermal responses of reinforced concrete beams during a fire. People today have a thorough understanding of the mechanical and thermal characteristics of steel and concrete at high temperatures thanks to decades of extensive research, which is now readily accessible. This section examines the existing finite element model of concrete behavior.

3.4.1 Concrete's Thermal Properties

Concrete's specific heat capacity and thermal conductivity are described in EN 1992-1-2 [15];

3.4.2 Elastic modulus of concrete

Concrete's specific heat capacity and thermal conductivity are determined by According to EN 1992-1-2 [15], the test sample's compressive strength determines the concrete's elastic modulus under different temperature variations. For all compressive-strength

concrete at all temperature levels, this model employs the same concrete Poisson's ratio according to EN 1992-1-2 [15].

3.4.3 Density Concrete

It is assumed that the density of concrete is always 2300 kg/m^3 .

3.4.4 Damage Plasticity Concrete

Concrete exhibits complex mechanical behavior at high temperatures, with distinct failure processes under tension and compression (cracking and crushing) and considerable nonlinearity.

In this FE model, an ABAQUS Standard damaged plasticity constitutive model is used to simulate the mechanical behavior of concrete. This model incorporates the concrete's compressive and tensile behaviors, which are covered in the sections that follow.

3.4.4.1 Compression behaviour of concrete

It is assumed that the response of concrete under compression is linear elastic until it reaches the initial yield surface. The hardening variable, a function of equivalent plastic deformation, governs the ensuing rise in the load surface. Thus, the load surface under multiaxial compression can be determined from the uniaxial compressive stress-strain relationship using the ideas of effective stress and equivalent plastic deformation.

The uniaxial compression stress-strain relationship of concrete at high temperatures is defined in this work using the Eurocode model. Up until the axial stress hits the initial uniaxial elastic limit, which is calculated to be $0.33 f_{c, T}$ (where $f_{c, T}$ is the uniaxial compressive strength of concrete at temperature T), it is believed that the compression response of concrete is linear elastic. The strain hardening curve, which reaches the maximum compressive stress, comes next, and following that is the lower limb, which illustrates the concrete's softening tendency.

3.4.4.2 Tensile behaviour of concrete

It is believed that concrete has linear elastic tensile behavior prior to fracture. To simulate the behavior of cracked concrete, the crack zone model and the elastoplastic constitutive model of the smeared crack method are coupled. The smeared crack model states that the fracture initiates upon reaching the designated yield surface, which coincides with the failure surface of the stress-dominated behavior. As a result, as the deformation (also known as tensile softening) develops, the tensile stress in the fracture zone gradually

diminishes. The element determines the expected deformation of the broken concrete in the smear crack model.

3.5 STEEL MODELLING

3.5.1 Thermal properties of Reinforced steel

The variation in thermal conductivity and specific heat capacity with the steel temperature as stated in EN 199312 [15] is adopted by this finite element model. Steel has a density of 7,800 kg/m³.

3.5.2 Model of plasticity for steel reinforcement

Two components make up the overall deformation of steel at high temperatures: stress-induced deformation (also known as the stress-tensile strain curve) and free thermal deformation (ϵ_{th}). Both are defined in the current finite element model, per EN 199212 [15].

3.6 Modelling of beam in ABAQUS

The ABAQUS program was used to model the 3D beam-reinforced concrete. For heat transport calculations, the concrete beam was modeled using the SOLID element. Temperature is the only degree of freedom it possesses at nodes. The software provided embedded steel reinforcement and concrete to achieve the full bonding that is assumed between the rebar and the concrete. 200 °C ambient room temperature was applied to the entire beam model as a predetermined initial condition. The beam is next exposed to fire at its vertical faces and soffits in the form of a typical time-temperature curve in accordance with ISO 834. In order to do this, the temperature boundary condition is defined at the beam's sides and soffit, and it fluctuates according to the transient heat transfer's amplitude on the standard time-temperature curve.

For support and loading, use a steel plate to achieve its objective by making the element compare to the sample beam. During modeling, use tie interaction between the beam and the steel plate. In addition to this, for loading purposes, steel loading is coupled to the surface of the load. For the support model, use a simple supported beam.

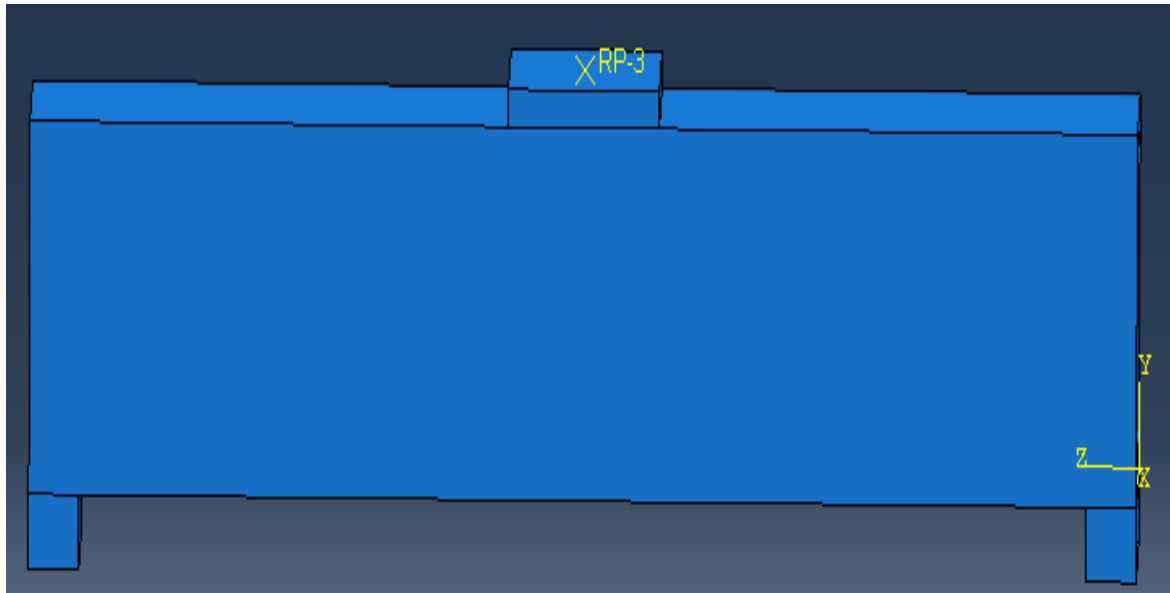


Figure 3. 2 Concrete beam layer FEM model

3.6.1 Reinforcement configuration

In this research, we used a doubly reinforced beam for all analyses that contained two bars by 8 mm diameter ($2\Phi 8$) for the compression zone and three bars by 8 mm diameter ($3\Phi 8$) for the tension zone to make the beam ductile. And provide 6 mm-diameter bars for the shear reinforcement within a 200-mm distance throughout the length.

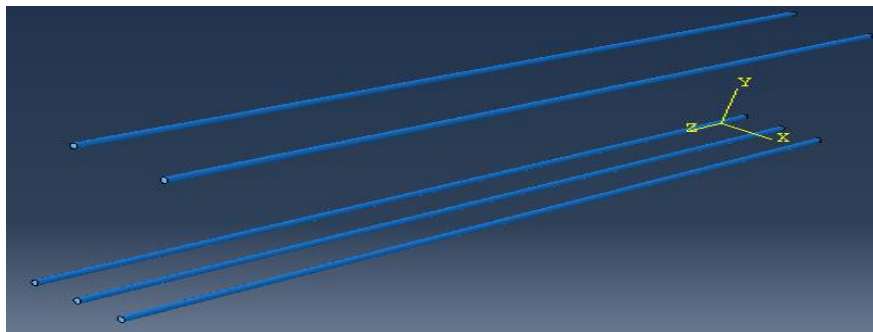


Figure 3. 3 Reinforcement bars FEM model.

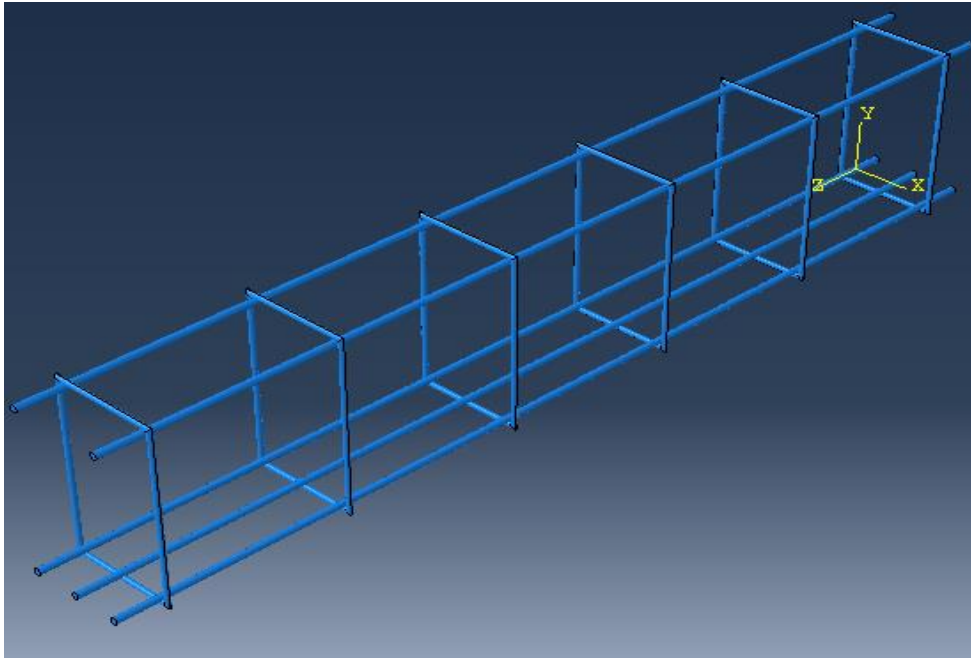


Figure 3. 4 Longitudinal Reinforcement bars and stirrups layer FEM model.

3.6.2 Mesh with Finite Elements

All of the model's elements were intentionally given the same mesh size in order to guarantee that node sharing occurs between two distinct materials and to produce correct results from the FE model. The model's chosen mesh type is structured. The mesh element C3D8R is a 3D solid mesh for concrete and reinforced steel.

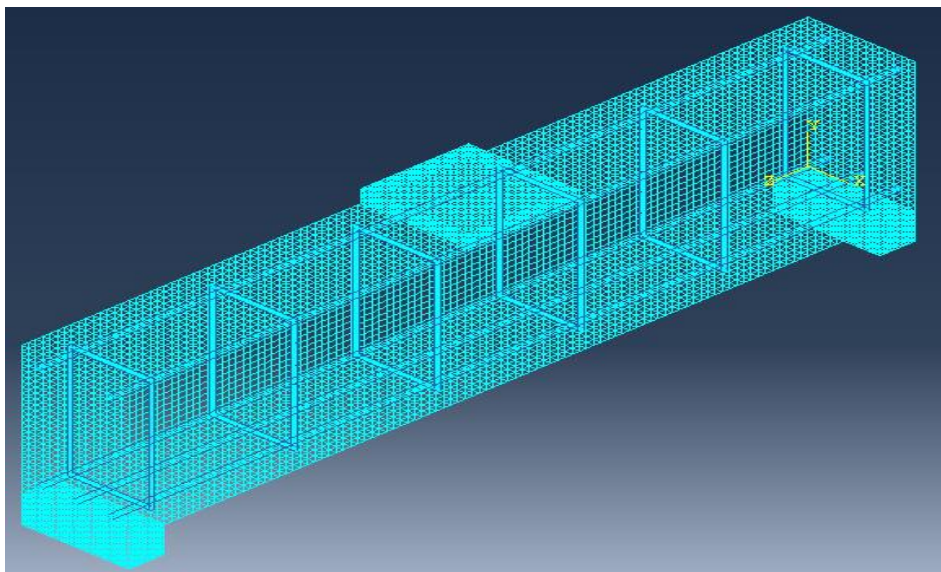


Figure 3. 5 Mesh configuration of Beam with Rebars

3.6.3 Framework for Assessing RC Beam Fire Performances

Since longitudinal heat transmission is ignored in the first step, the temperature distribution over time is calculated using a transient thermal analysis on a 2D FE model of the beam cross-section.

A flowchart of the suggested approach is provided to provide a detailed description of the numerical analysis process. A 2D FE model is first used to assess the specimen's thermal distribution in the event of a fire. Then, as the temperature distribution stays constant, the external forces are applied to the RC beam in progressively larger loading stages. As a result, the issue of an RC beam's fire performance has been moved to structural analysis, where a new stress-strain relationship updated by a given thermal distribution is used. The suggested one-dimensional spectral model will then be used to forecast the RC beam's mechanical behaviors.

When the applied loadings in the structural analysis are low, the concrete and reinforcement elastic moduli can still be calculated. But as the stress increased, the materials' nonlinear behaviors would manifest. Equivalent secant elastic moduli for the steel reinforcement and concrete are used to model this occurrence; these moduli would be changed simultaneously with the local strain. Throughout the updating procedure, the empirical Eurocode formulas for every temperature scenario supplied by the prior transient thermal analysis are used to determine the temperature-dependent constitutive relations. Since perfect bond performance is assumed in this analysis, as was previously noted, the strain of the steel bar equals the strain of the concrete at the same place. This means that until force equilibrium is attained, the DSM is implicitly updated to replicate the concrete's gradual breaking throughout iterations. As a result, the structure's displacement vector is determined for a single fire scenario, and the process can be repeated for the subsequent thermal stage.

The calculation process is carried out implicitly to obtain the nodal displacement vector and the associated load-carrying capacity via the iterative procedure until convergence is obtained, in accordance with the numerical approach shown in Figure 3.6. In this way, the load on the RC beam can be gradually increased until failure occurs following fire exposure, allowing the structural reactions to be assessed. As an alternative, the development of time-temperature exposure can also be used to identify the mechanical behaviors under a particular load.

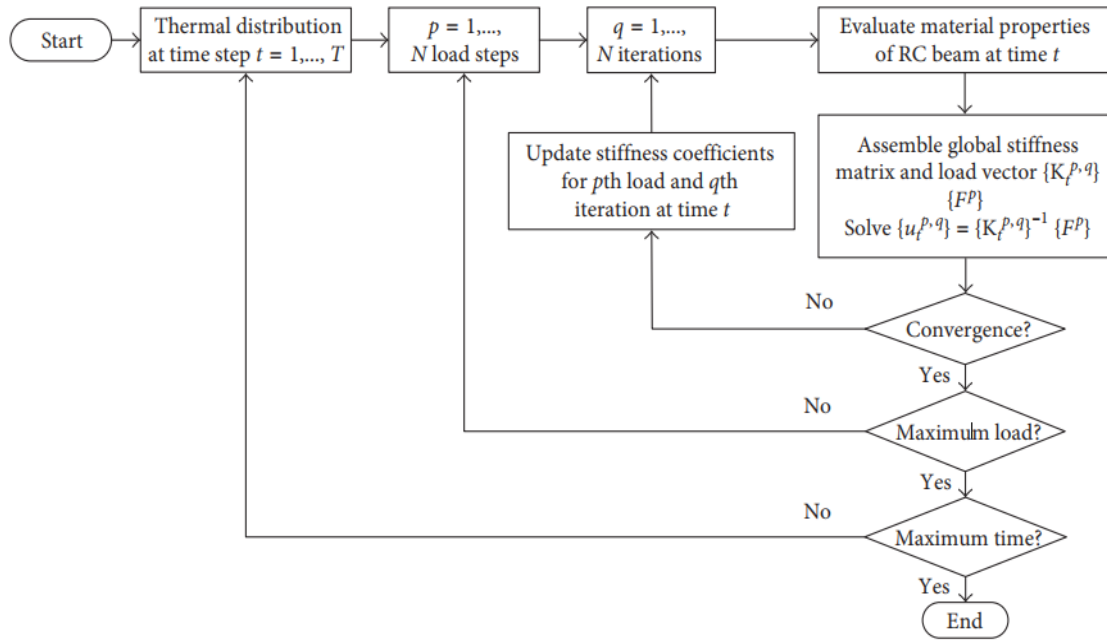
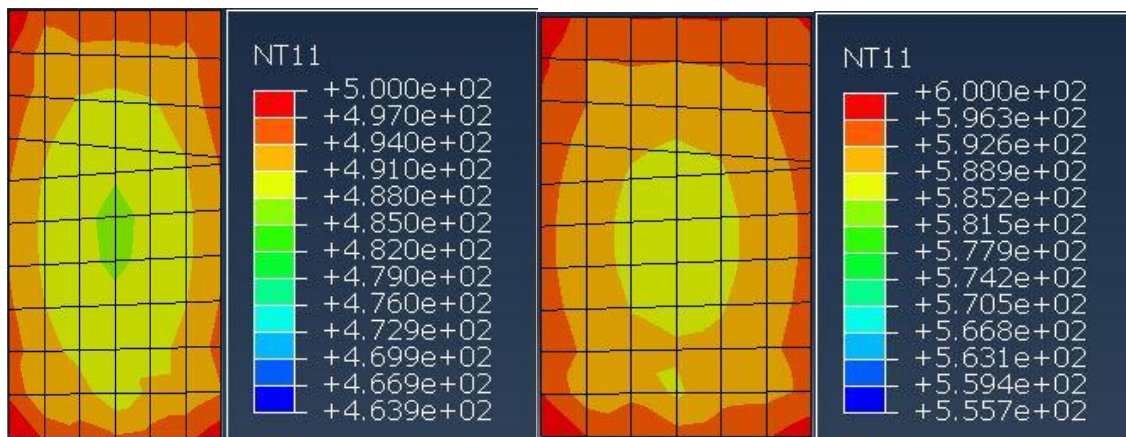
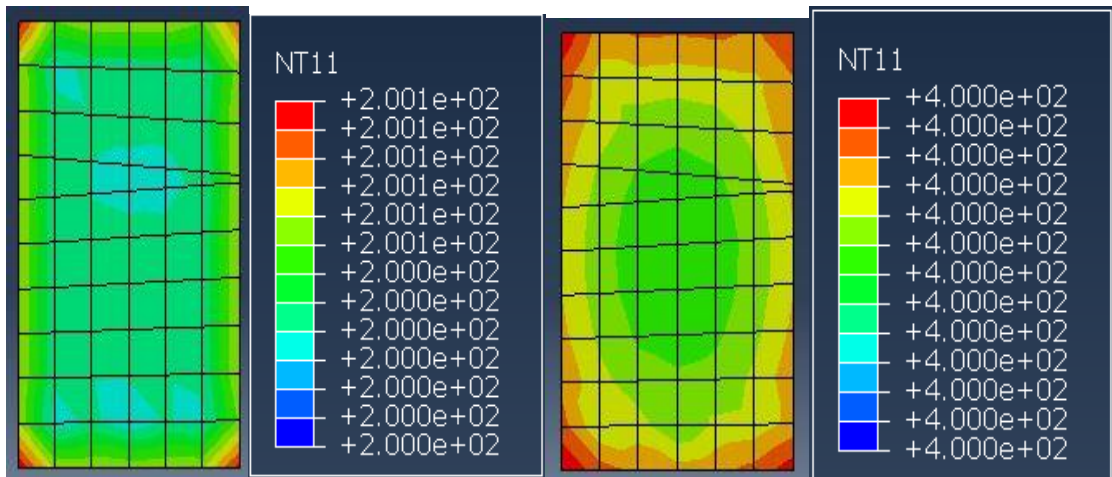


Figure 3. 6 The suggested numerical strategy's framework



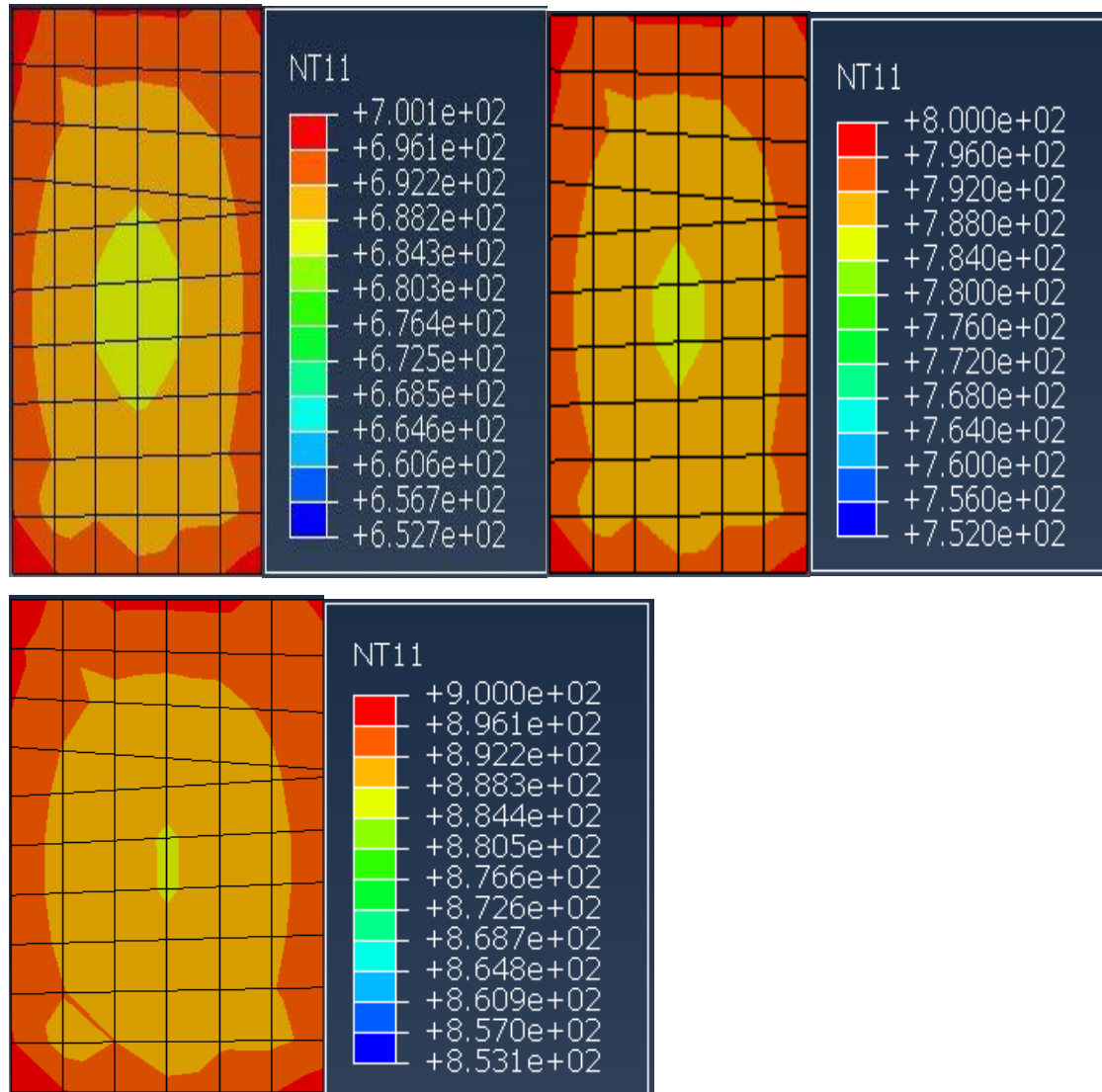


Figure 3. 7 Temperature distribution through model

Chapter 4 RESULT AND DISCUSSION

4.1 Finite Element Analysis Result

4.1.1 Result of 25 Mpa and 30 Mpa Cube compressive strength on fire
Cube Compressive strength 25Mpa and 30Mpa after cooling resulting in different durations of time are listed below figure:-

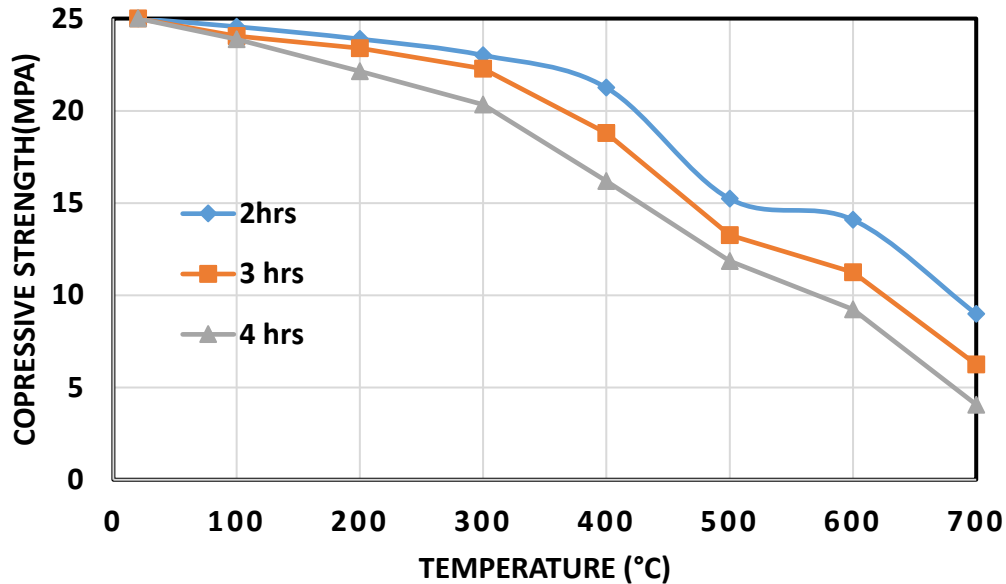


Figure 4. 1- Impact of fck-25mpa on concrete's compressive strength during varying fire durations.

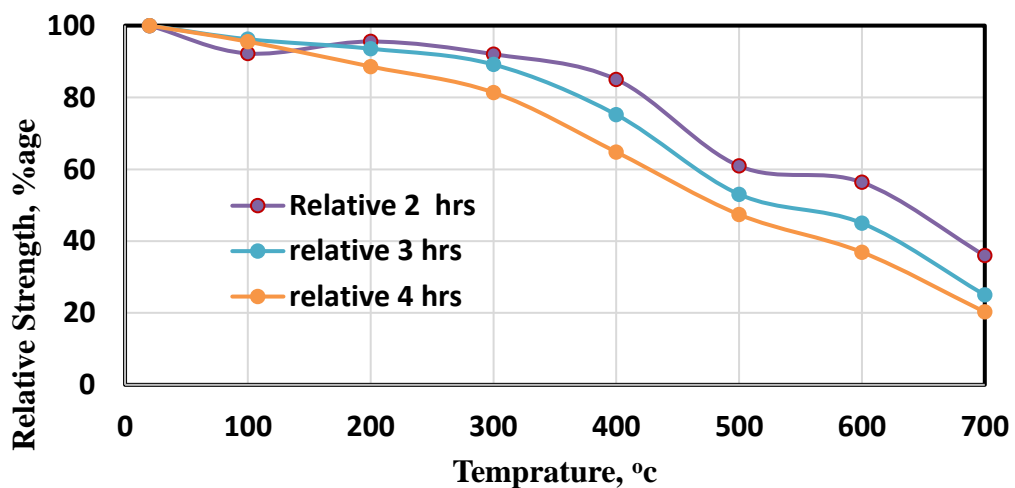


Figure 4. 2 Relative strength Fck-25Mpa cube strength loses Different fire duration.

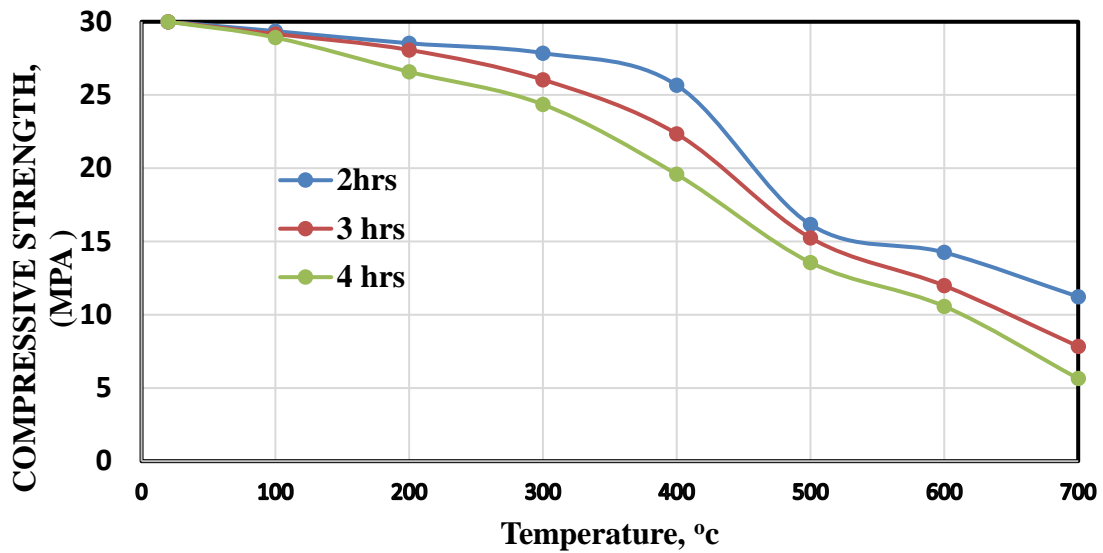


Figure 4. 3- Impact of fck-30 mpa on concrete's compressive strength during varying fire durations.

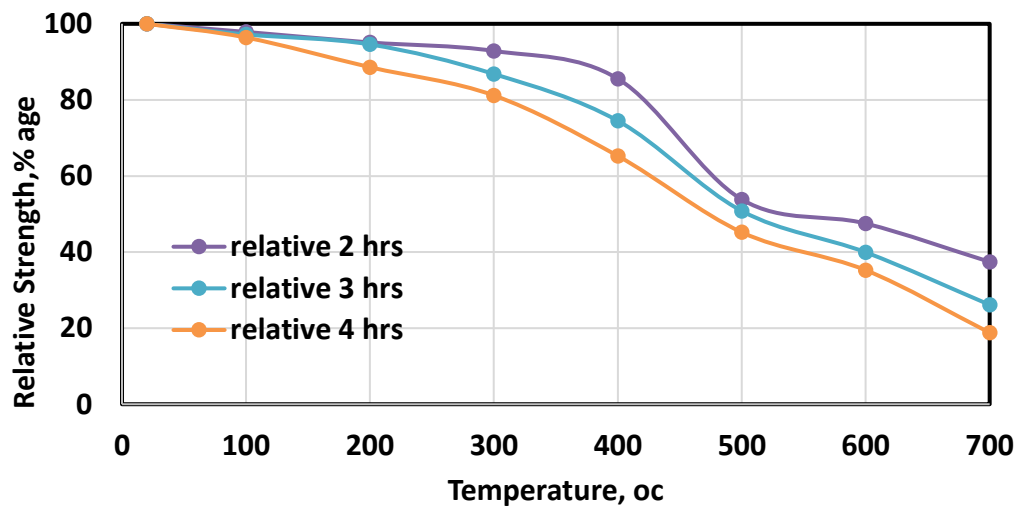


Figure 4. 4- Relative strength FCK-25MPA cube strength loses different fire duration.

4.1.2 Result of the Beam with 15mm and 25mm concrete cover

For a beam that is simply supported but has a distinct concrete cover, that means 15mm and 25mm concerning the duration of fire, with 25MPa and 30MPa listed below.

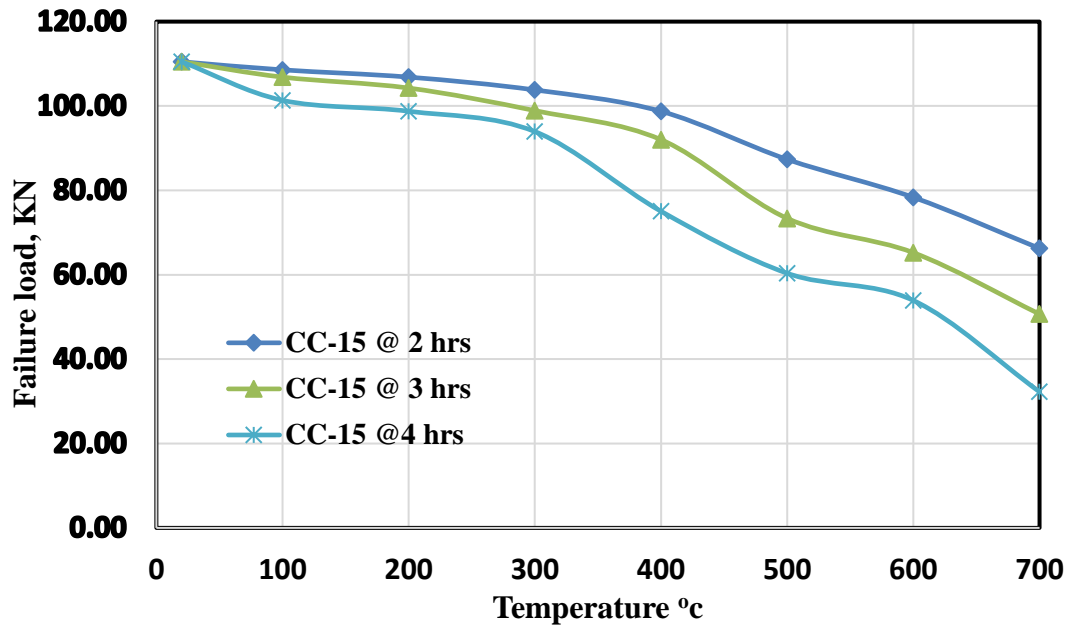


Figure 4. 5- Effect failure load of 15mm concrete cover at the different duration of the fire.

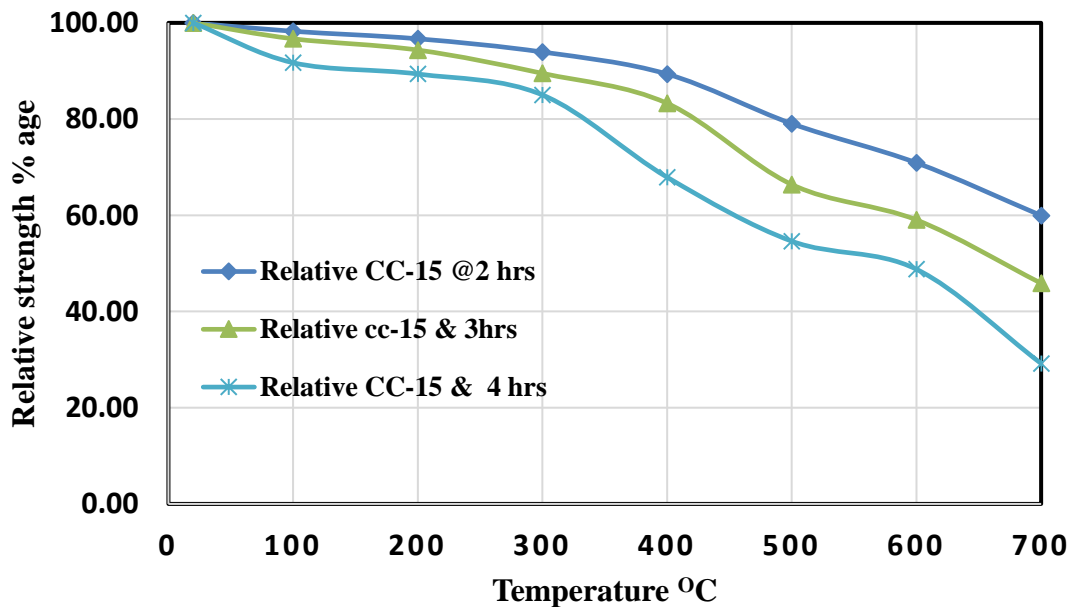


Figure 4. 6- Relative strength of a beam at 25 MPa with a 15 mm concrete coating at various times

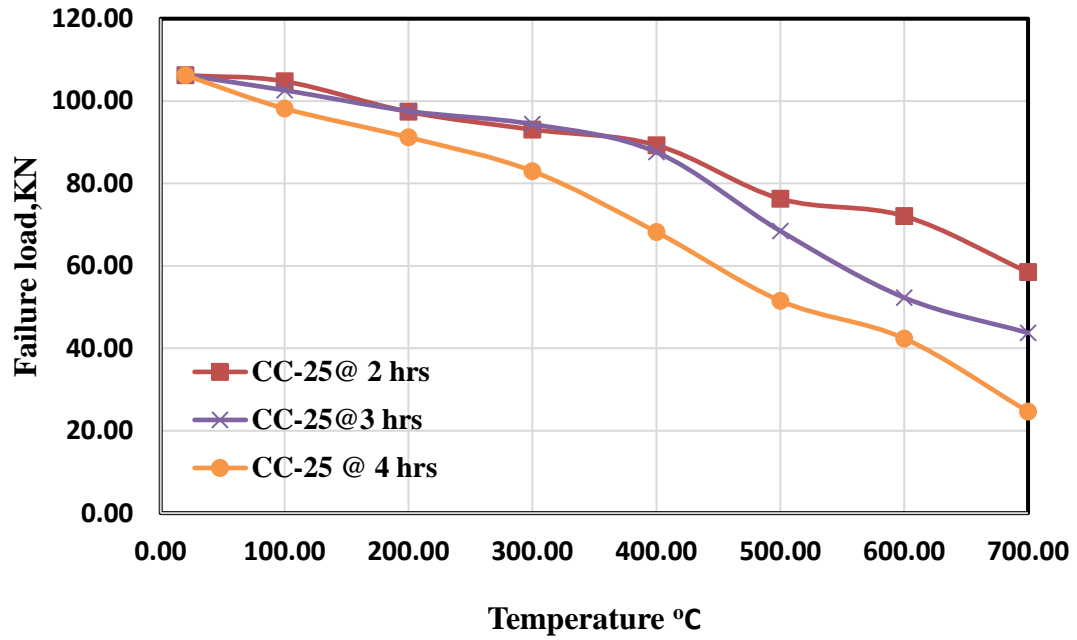


Figure 4. 7- Effect failure load of 25mm concrete cover at the different duration of the fire.

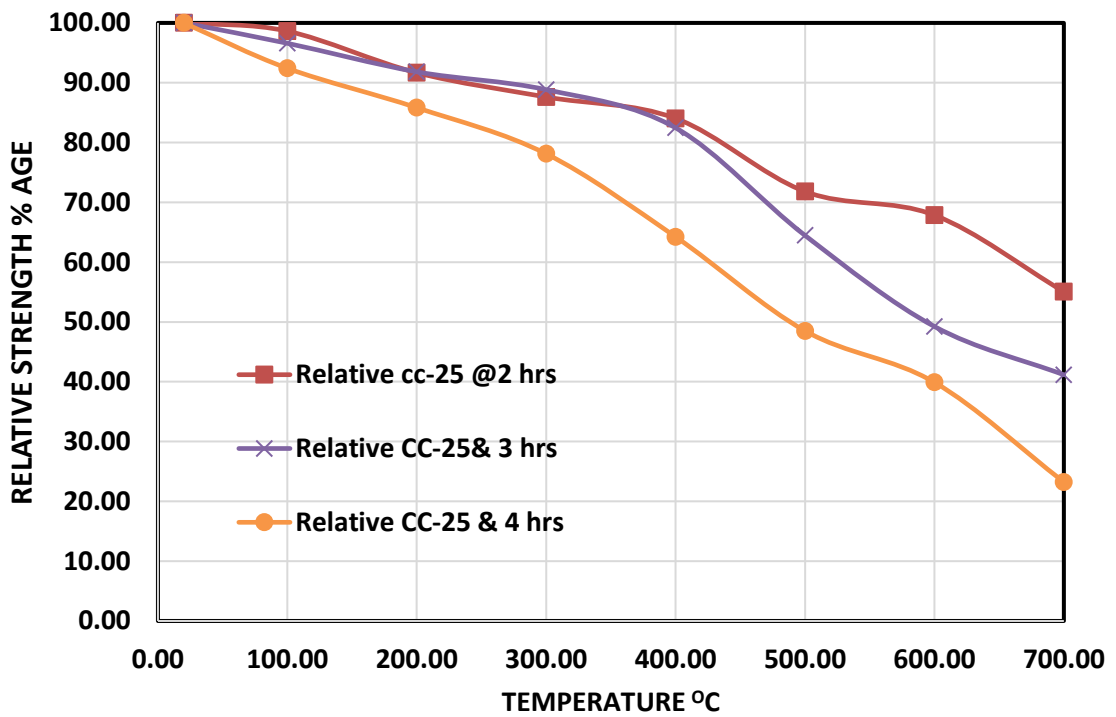


Figure 4. 8 Relative beam strength with a 25 mm concrete cover over various time intervals using 25 MPa.

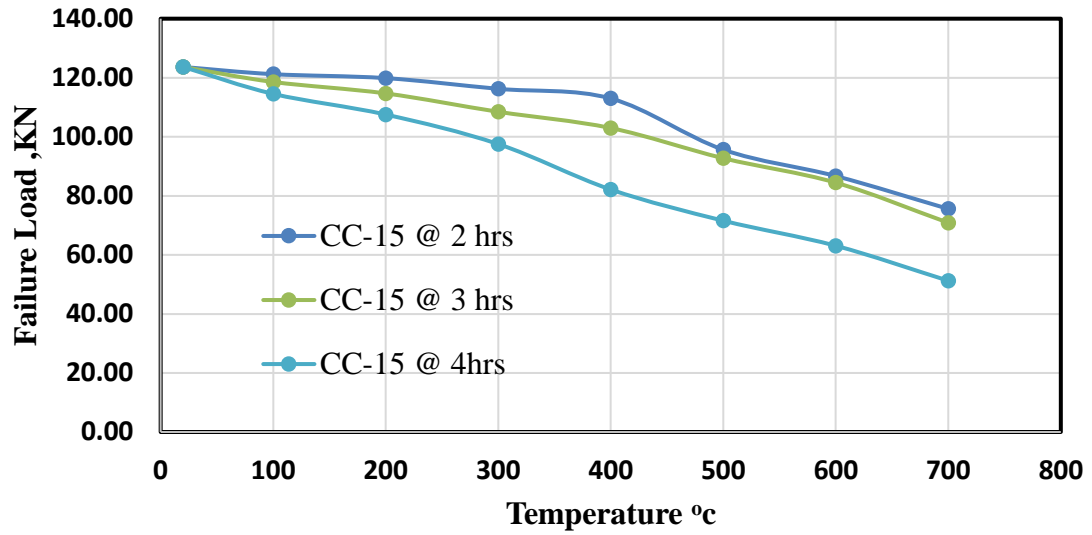


Figure 4. 9 Effect failure load of 15mm concrete cover at the different duration of fire at 30Mpa.

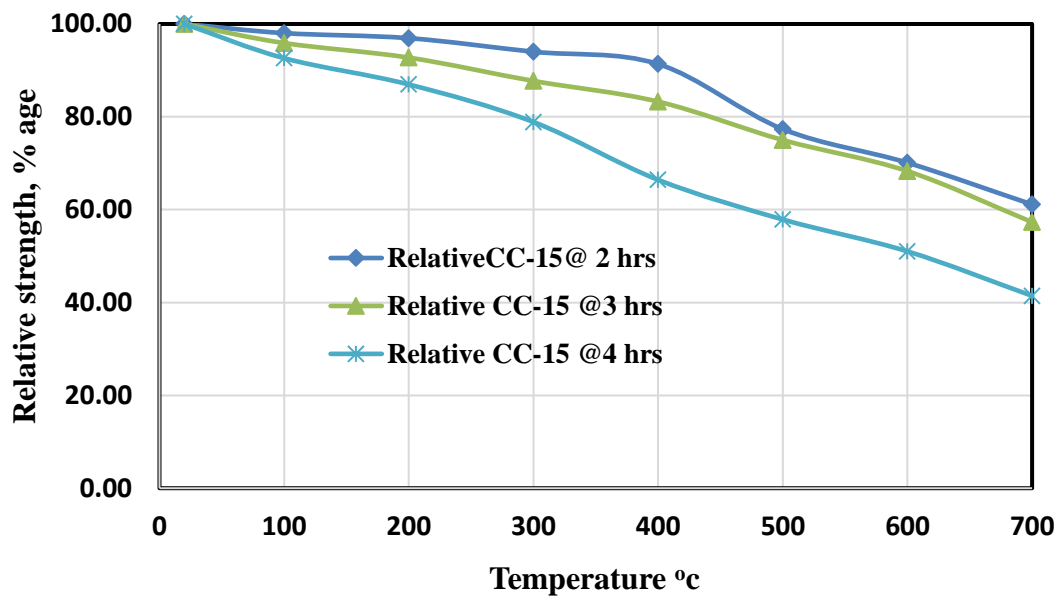


Figure 4. 10- Relative strength of the beam with a 15 mm concrete Cover over various times at 30 MPa

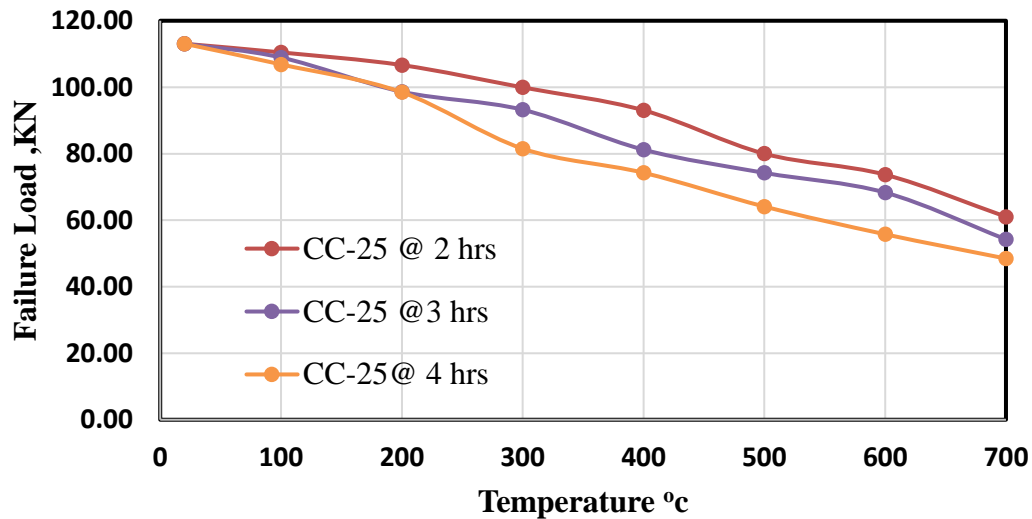


Figure 4. 11- Effect failure load of 25mm concrete cover at the different duration of fire with 30Mpa.

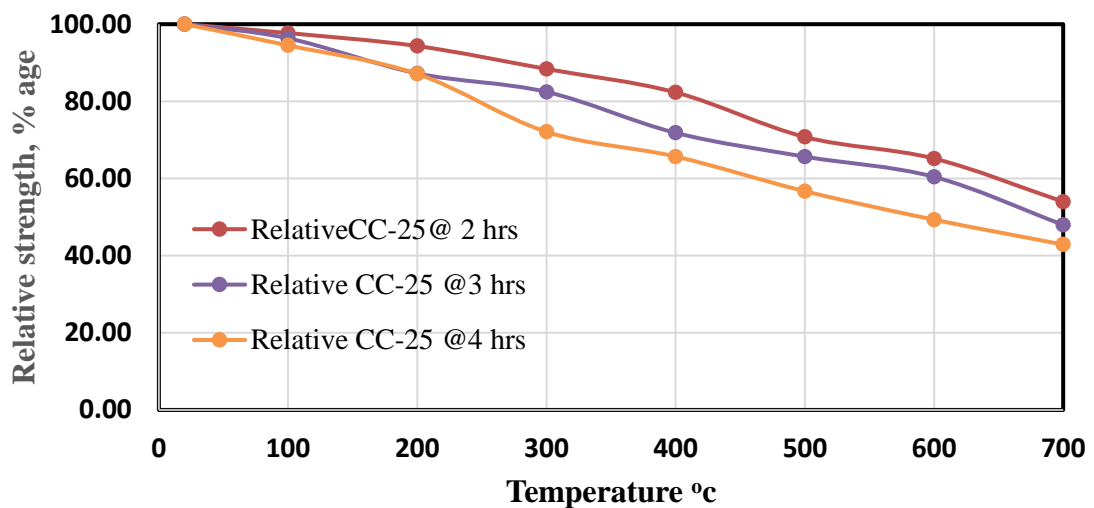


Figure 4. 12- Relative strength of the beam at 25 mm concrete cover with 30 MPa over various times.

4.2 Results and Discussion

4.2.1 25Mpa and 30 Mpa Cube Compressive Strength Exposed To Fire

This paper examines the impact of concrete's compressive strength at different fire intensities and durations. 150mm cube size was used to simulate Abaqus with firing at two

hours, three hours, and four hours. The test was done after cooling at room temperature (20 °C).

In the experiment shown in Figures 4.2 and 4.4, the impact of fire on concrete's 25 MPa compressive strength dramatically decreased as the temperature rose to 500 °C. Because the compressive strength of concrete diminishes as fire intensity increases, the length of the fire is also closely proportional. The relative percentage of compressive strength at 700 °C with two hours, three hours, and four hours observed in Figs. 3.3 and 3.5 is presented below in the table.

For 25 MPA Compressive strength

Table 4. 1- Fck-25Mpa at 700 °C

Item	T °C	TIME	FCK(Mpa)	% age
1	700	2	8.99	35.96
2	700	3	6.25	25
3	700	4	4.07	20.28

For 30 MPA Compressive strength

Table 4. 2- Fck-30 MPA at 700 °C

Item	T °C	TIME	FCK(Mpa)	% age
1	700	2	11.23	37.43
2	700	3	7.84	26.13
3	700	4	5.64	18.8

From tables 3.1 and 3.2, the relative percentage of compressive strength losses at 700 °C resulted in 80% losing its compressive strength at 4-hour firing in the cases of 25 MPa and 30 MPa.

4.2.2 Simply reinforced concrete beams result in different parameter

The size of the beam used to test for fire is the same dimension that is used by habtamu, which means B/D/L (150mm, 250mm, and 1100mm). This test uses a concrete cover of 15mm and 25mm with a 2-4 hour interval of one hour at a 100°C–700°C interval of one hundred degrees.

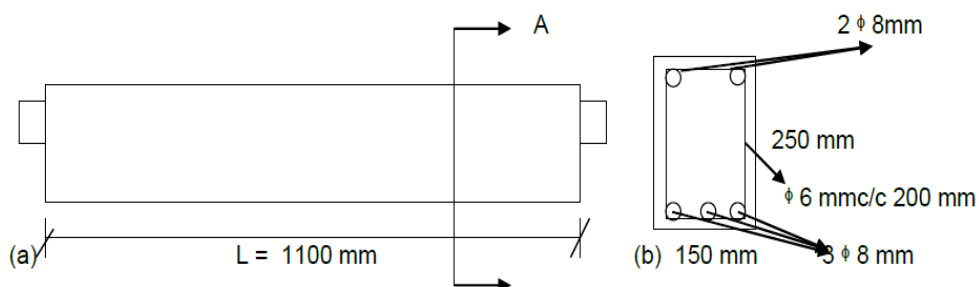


Figure 4. 13- (a) - Sample RC beams' longitudinal section (b) and cross-section are shown in reference [32].

From the analysis, results observed from figures 4.6 and 4.7 indicate that 25 Mpa compressive strength exposed to fire with varying durations is indicated at a temperature of 100°C–200°C, which has no significant effect on the failure, but as the temperature increases and reaches 500°C, the failure load is reduced to 50–60%. There is one thing that is not certain: the effect of the concrete cover after cooling the reinforcing bars to gain their strength. To determine the effect, the loading must be done during the firing of the beam specimen. Due to this effect, the result is not much different for concrete covers of 15mm and 25 mm.

However, because the temperature is still relatively low (around 100 °C), the mechanical characteristics of steel and concrete remain mostly constant at this stage. As a result, the anticipated fire performance of the RC beam is not much affected by the temperature underestimation. The relative losses of the 25 Mpa beam burning at 700 °C result from 75–85% of the losses of failure load.

4.3 THE PROPOSED FINITE ELEMENT MODEL'S VALIDATION

4.3.1 CUBIC COMPRESSIVE STRENGTH OF CONCRETE

To demonstrate the potential and precision of the current FE model, cube compressive strength and RC beams subjected to fire testing by Habtamu [32] were chosen and examined. These tests were chosen because comprehensive reports of their outcomes made it easier to do FE simulations and thorough comparisons. The ISO 834 standard fire was applied to the beam during the heat transfer analysis.

To compare the 25 MPa cube compressive strength of concrete with an experiment done by Habtamu [32] at 3 hours and 4 hours duration.

Comparison of 25 Mpa cubes' compressive strength after burning

Table 4. 3- In contrast % of losses of compressive strength of 25Mpa after burning

T (°c)	Time (hrs)	compressive strength, Mpa (Experiment)	compressive strength, Mpa (FEM)	%age loss(Experiment)	%age loss(FEM)
20	0	27.03	25	108.12	100
100	3	22.85	24.05	88.9752	96.2
200	3	28.28	23.39	86.5335	93.56
300	3	21.79	22.29	82.4639	89.16

20	0	27.03	25	100	100
100	4	21.85	23.88	88.3463	95.52
200	4	24.30	22.15	81.946	88.6
300	4	18.34	20.34	75.2497	81.36

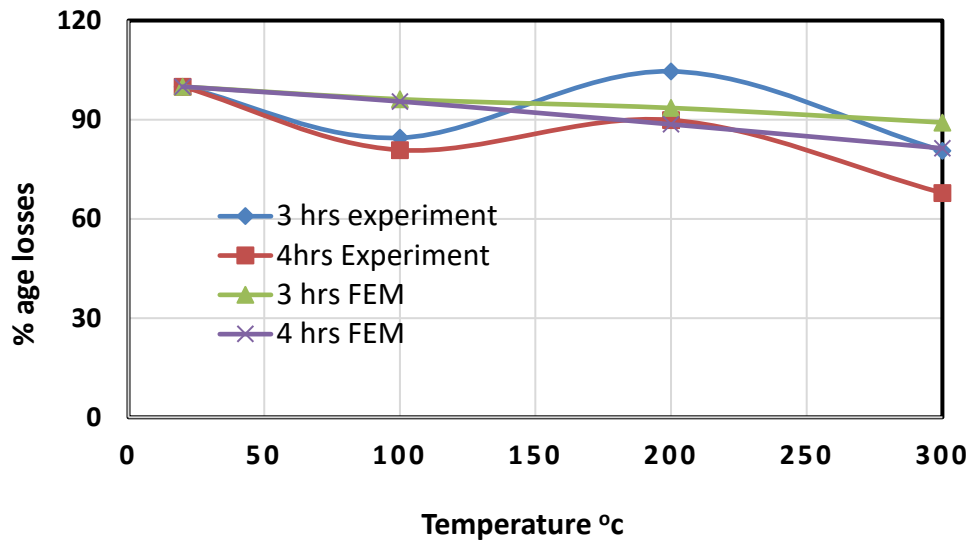


Figure 4. 14-Comparison of 25 Mpa cube compressive strength % age losses after burning

The experimental result is done by Habtamu [32], and FEM gives a close argument with a small error. The error comes during the experiment with the concrete cube because the result shows a temperature reaching 200 oC. The unburned compressive strength is less than the resultant concrete compressive strength. This phenomenon happens if the sample is not properly mixed or it needs further study, but the other simply gives a reasonable result compared to the experiment done by Habtamu. Comparison of 30 MPa cube compressive strength after burning.

Table 4. 4- Comparison of %age losses of compressive strength after burning

Target Temperature, °c	Time (hrs)	compressive strength, Mpa (Experimental)	% age loss(experimental)	compressive strength, Mpa (FEM)	% age loss(FEM)
20	0	43.4	100	30	100
100	3	35.36	81.48	29.16	97.21
200	3	35.38	81.51	28.08	94.59
300	3	32.94	75.91	26.04	86.81
20	0	43.4	100	30	100
100	4	37.62	86.68	28.92	96.4
200	4	37.58	86.59	26.58	88.6
300	4	29.32	67.56	24.35	81.18

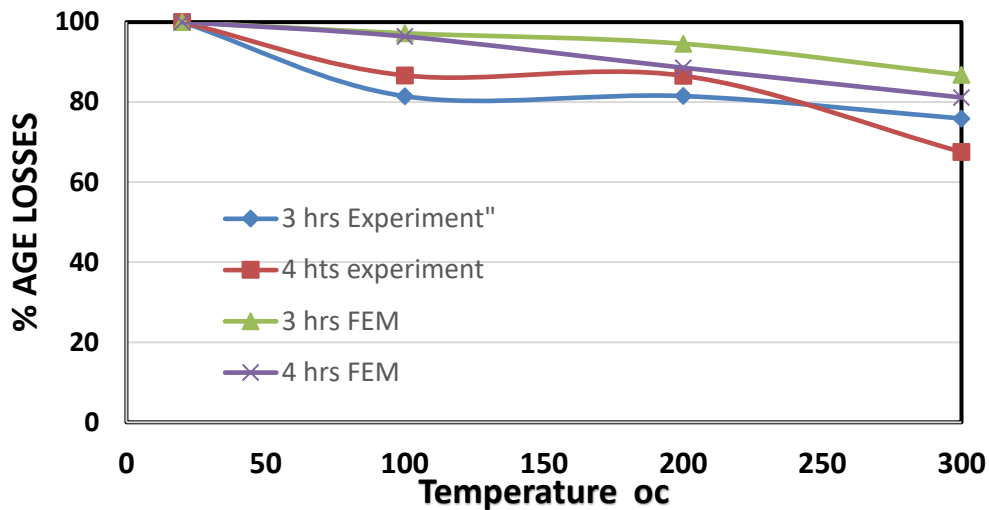


Figure 4. 15- Comparison of 30 Mpa cube compressive strength % age losses after burning.

4.3.2 REINFORCED CONCRETE BEAM MODEL

The proposed numerical method for structural analysis of fire-exposed RC beams was shown to be accurate and capable by referring to the experimental tests conducted by Kumar and Kumar [34]. A series of reinforcing concrete beams were subjected to varying fire exposure times in Kumar's experiments in order to examine their structural behavior. The dimensions of the reinforcing beam are 200 mm for width, 300 mm for depth, and 3950 mm for length. The spacing between the supports is 3200 mm. With a shear span of 800 mm, the four-point load is progressively applied to the beam; Figure 4.16 shows the experimental setup.

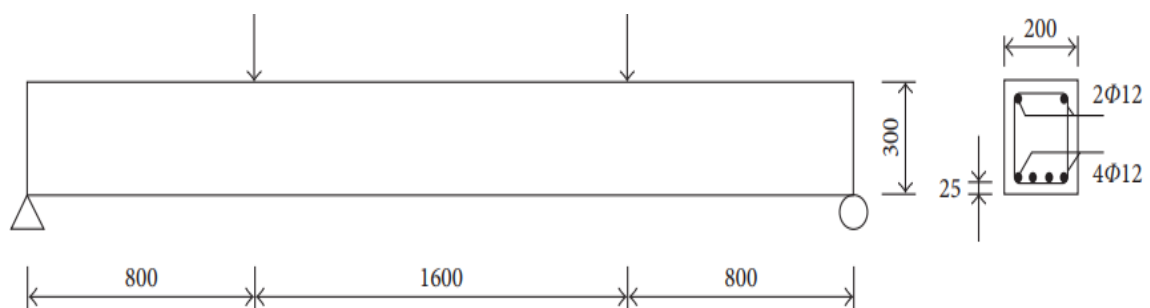


Figure 4. 16 Experimental test setup

Table 4. 5 Material and geometrical properties of specimens

Concrete	Elastic modulus (MPa)	19,600
	Compressive strength (MPa)	17.1
	Tensile strength (MPa)	2
Steel bar	Diameter (mm)	12
	Yield stress (MPa)	480
	Ultimate strength	550
RC beam	Span (mm) 3200	3200
	Width (mm) 200	200
	Height (mm) 300	300

The bottom half and two sides of the beam were subjected to the following temperature-time relationship, while four samples were chosen for fire testing in a furnace with a constant ambient temperature of 20°C. In line with the ISO 834 standard:

$$T(t) = T(t_0) + 345 \log(8t - 1),$$

where the starting ambient temperature (20°C) is denoted by $T(t_0)$. They spent one, one and a half, two, and three hours in the fire. During the sample cooling process, the ambient temperature was kept at 20°C. Four-point load tests were conducted on the samples, one of which was a reference beam that was not heated to the point where strength testing was not feasible. their inconsistency. During the test, the RC beam's deflection was managed by means of two load plates. Due to concrete collapsing after cooling, testing beams exposed to fire for 2.5 hours was not practicable, so only experimental data are provided for 4 samples (reference beam and beam exposed to fire for 1, 1.5, and 2 hours).

The more complicated FE model's numerical prediction and the experimental findings match fairly well. As a result, they will be employed to verify that the streamlined spectral model functions as intended.

4.3.2.1 Results and Discussion.

This study is divided into two primary phases: the structural phase and the transient thermal analysis phase, as was mentioned in the previous section. Because the bottom and two sides of the beams were exposed to high temperatures during the fire test, FEM analysis can be used to assess the RC beam's heat distribution at each time step. The heat load was applied to the bottom face and both sides of the 60 elements that were created along the cross-section in depth using the 5 mm element size in the FE model. This allows one to assess the temperature distribution over the whole cross-section.

Thermal data from experimental tests are not included in [34], which makes it necessary to compare the numerical thermal findings with measured experimental data in order to verify the accuracy of the suggested 2D FE model. Because the RC beams utilized in [34] and [33] have relatively comparable geometric and material features, the experimental thermal findings measured in [33] were used to compare the numerical thermal results in this work in order to solve this problem. As a result, Figure 4.17 compares the numerical and experimental results in the intermediate layer of RC beams at various depths of 10, 25, and 100 mm. It finds that the numerical results are in good agreement with the available test results.

The temperature distribution along various depth levels during 0.5, 1, 1.5, and 2 hours of fire exposure is displayed in Figure 4.18 to help better understand the thermal response of the RC beam. The numerical results are consistent with information found in the literature [34]. By averaging the temperatures of the element's four nodes, one may determine the temperature in each block. The collected thermal data can be used to determine the nonlinear stress-strain relationship between the concrete and steel reinforcement at any point on the cross-section. Next, structural analysis is carried out using numerical simulation and the suggested spectral model in accordance with the framework shown in Figure 3.6.

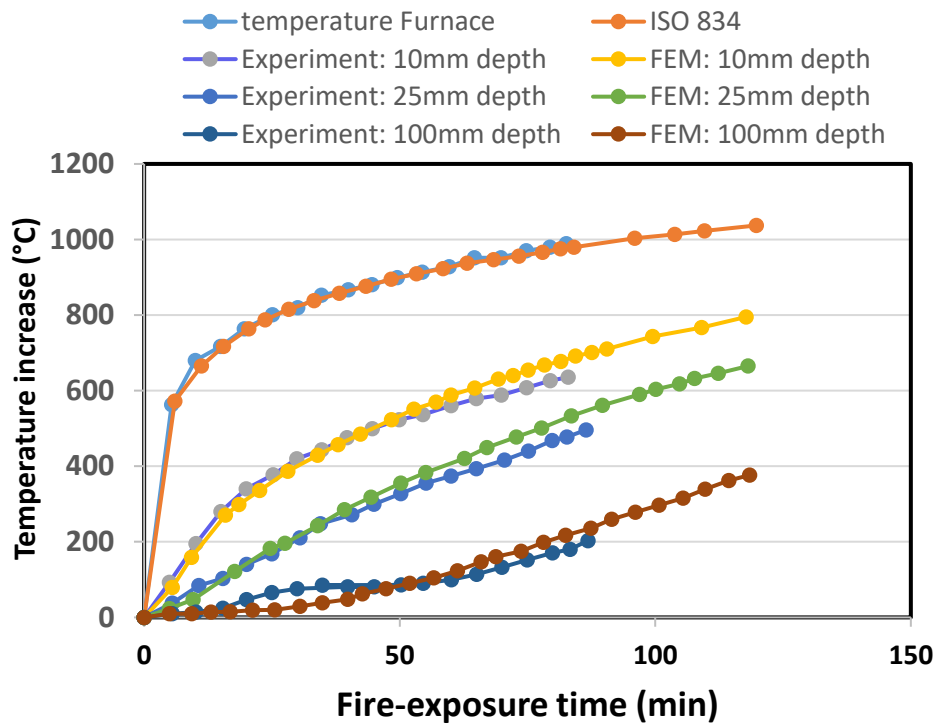


Figure 4. 17 Temperatures measured using FEM at different points throughout the fire.

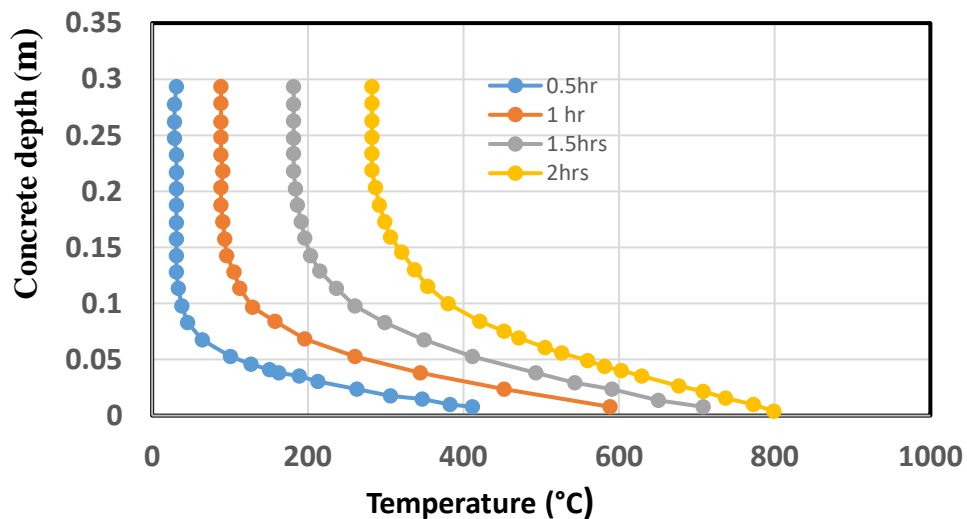


Figure 4. 18 Temperature distributions at the center layer of the midspan with fire duration over the concrete depth

The proposed spectral model states that if there are no discontinuities in the geometric and material properties across the portion, then one element is adequate to represent it. Because more spectral elements can be used to further simulate the RC beam's gradual attenuation due to the material's nonlinearity, more accurate results will be obtained in this study to capture the mechanical properties of RC beams with increasing loads under various fire situations. suitably via a finer mesh. The accuracy of the numerical results will increase

with the application of more elements, but this increase might not be sufficient to offset the notable increase in computing effort. Thus, depending on the required level of precision, the number of components can be found by striking a compromise between accuracy and computing cost.

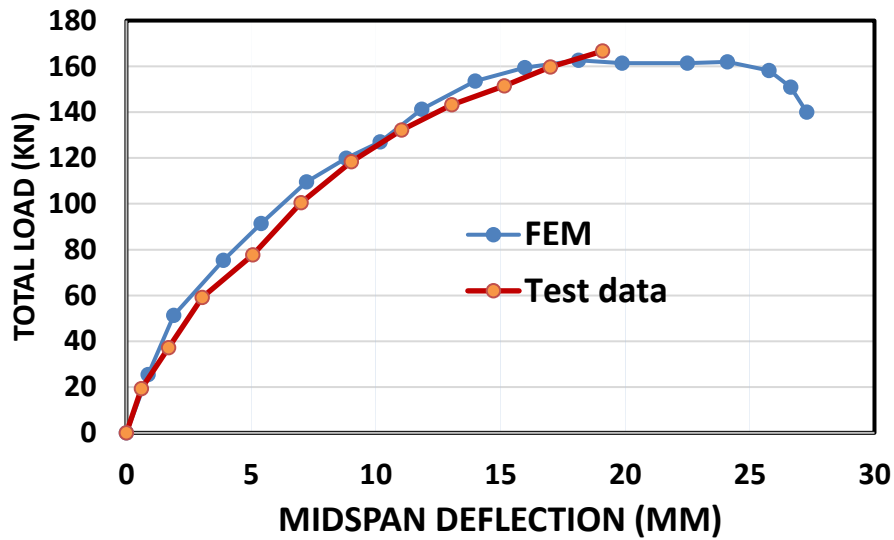


Figure 4. 19 Deflection reference beam for load-midspan

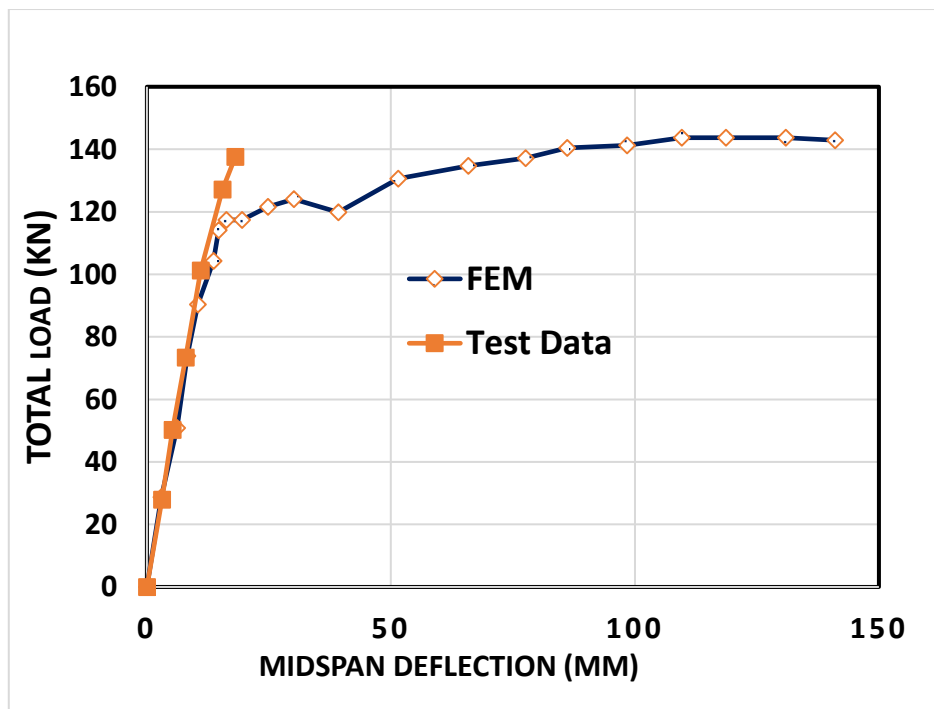


Figure 4. 20 Load-midspan deflection of 1 hr fire duration

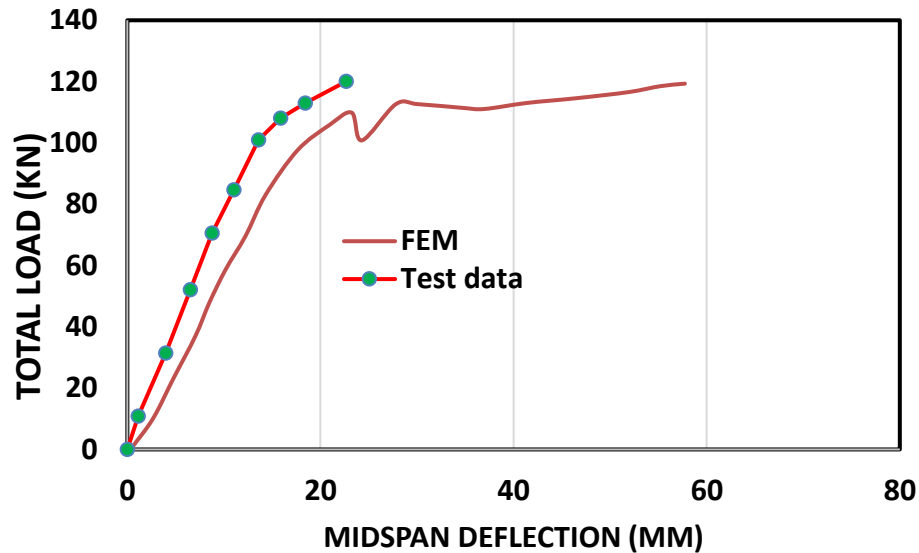


Figure 4. 21 Load-midspan deflection 1.5hrs fire duration

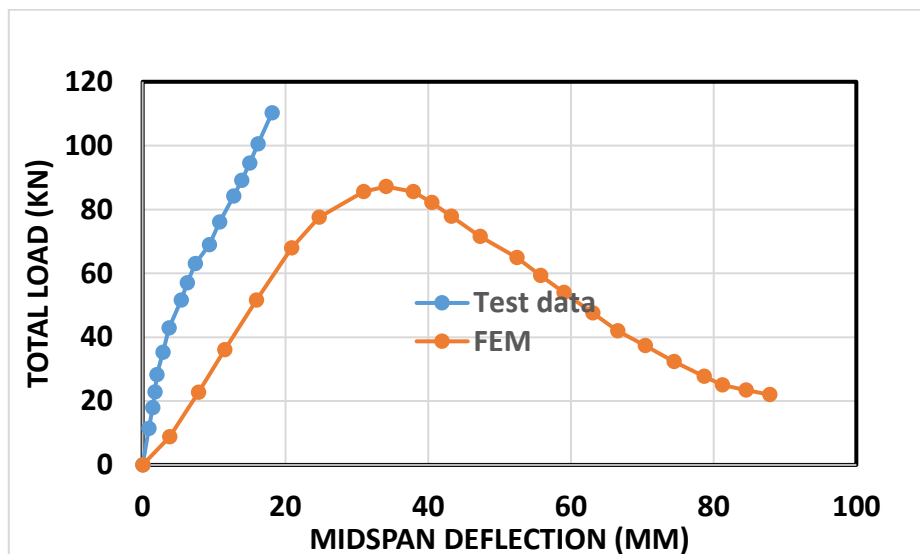


Figure 4. 22 Load-midspan deflection 2 hrs fire duration

Fire duration For the structural analysis, the suggested spectral model's 200 mm element size is chosen, so this study uses 16 spectra to simulate the mechanical behavior of a 3.2 m-long RC beam. Only certain elements are available. This significantly reduces the computational complexity compared to the traditional finite element model. The numerical results of the RC beam for four fire situations (no heating and heating after 1, 1.5, and 2 hours) are displayed in Figures 4.19 to 4.22, and the results are compared to confirm the accuracy of the suggested numerical approach. Figure 4.19 displays the reference beam's load-deflection curve. The FE model and the experimental data accord with the numerical results rather well. The steel falls and the concrete fractures at load points of 50 and 150

kN. As demonstrated in Figures 4.20 to 4.22, the cracking load of the RC beam dropped to 40, 35, and 30 kN after one hour, 1.5 hours, and two hours, respectively, in accordance with the numerical results predicted by the suggested model. This is assumed to be caused by the fact that concrete becomes less strong as temperature rises.

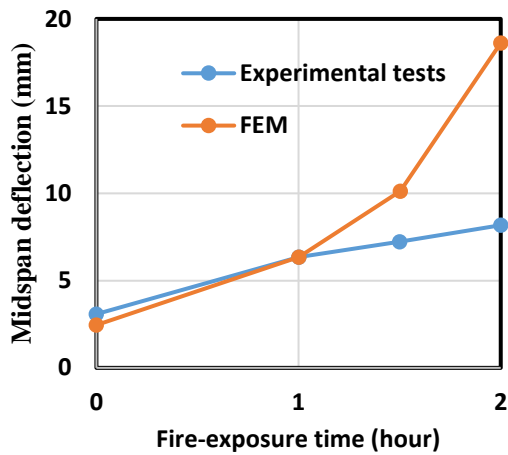
In the three fire situations, the steel's yield strength is 140, 110, and 70 kN, demonstrating a loss in strength brought on by thermal stress. The FE model in Figures 4.19 to 4.21 and the experimental testing agree with the numerical results. The suggested experimental model and the FE model diverge significantly after two hours of fire exposure, as Figure 4.22 illustrates.

This is explained by the fact that the behavior of the interfacial bond between concrete and reinforcing steel, which greatly affects the overall performance of the structure at high temperatures, can be accurately modeled using the Finite Element Methodology. For the purposes of this study, it is assumed that the concrete and reinforcing steel are fully bonded. This assumption can be used to forecast how RC beams subjected to fire will deteriorate in terms of load capacity.

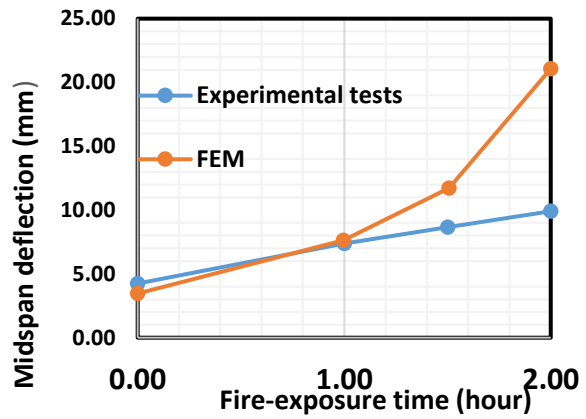
It is crucial to take into account that a decrease in material strength is a significant element, and the suggested one-dimensional numerical model is designed to carry out and streamline the structural analysis in accordance with this. However, as one of the primary goals is to effectively represent the mechanical behavior of the RC beam under fire exposure, the accuracy loss can be offset by the effectiveness of the suggested spectrum model.

The suggested spectral technique uses the equivalent secant elastic modulus, which is updated implicitly throughout the course of the iterations, to introduce the material's nonlinear behavior. As illustrated in Figure 3.6, in order to prevent problems with convergence, we split the external loads into load levels and use an iterative secant approach to find a solution until force equilibrium is reached at each load level. As long as the applied load is greater than the load capacity of the RC beam, no convergence is obtained since this load management approach causes the nodal displacement to grow with increasing load. Thus, Figures 4.19 to 4.22 show a monotonic rise in the load capacity as a result of this study's use of the load control technique. However, the precision of the numerical findings is deemed sufficient because the suggested model only uses a restricted

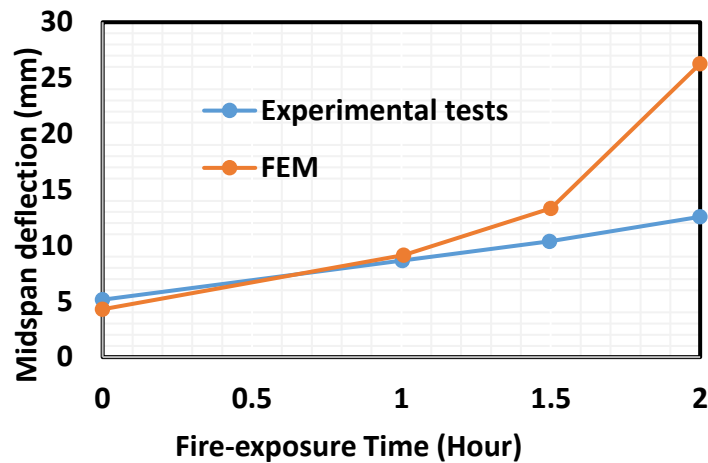
number of elements. As a result, the suggested spectrum model can be used to ascertain how fire-exposed RC beams will behave mechanically. In order to enhance comprehension of the structural deformation of reinforced concrete beams following fire exposure, Figures 4.23(a) through 4.23(c) below illustrate the mid-span deflections under external loads of 60, 70, and 80 kN, as shown below.



(a) 60 kN



(b) 70 kN



(c) 80 kN.

Figure 4. 23 Under the following external loadings: (a) 60 kN, (b) 70 kN, and (c) 80 kN, the fire exposure time-midspan deflection.

Large displacements at the span's center are seen in the experimental data as fire exposure increases; these displacements range from 3.1, 4.2, and 5.1 mm after 0 hours of fire exposure to 8.1, 10, and 12.5 mm after 2 hours. Exposure to fire rises by three hours. The results of the FEM analysis and the suggested numerical method showed, as predicted, the same behavior, showing a decrease in the resistance of the RC beams under heat loading. After 1.5 to 2 hours of fire, a sharp rise in mid-span deflection is seen, based on the suggested experimental model and finite element method. This is mostly due to the fact that, as Figure 4.23 a and b illustrate, steel loses strength quickly at 600 °C. After 1.5 to 2 hours of fire exposure, the temperature at the site of the tensile reinforcement (located 25 mm from the bottom), as depicted in Figure 4.18, surpassed this value. Large deformations can also result from heat-induced damage to concrete or reinforcing steel, as Figure 4.23 illustrates. It is commonly recognized that the temperature distribution derived from the transient thermal analysis has a significant impact on the mechanical response of reinforced concrete beams.

Thermal findings are produced by utilizing 2D thermal analysis, and typical cross-sections of RC beams are selected without taking longitudinal heat transport into account. This simplification suggests that inaccuracies from earlier thermal analyses may be the reason for the difference between numerical predictions and experimental results, particularly when there has been an extended fire exposure. Following two hours of exposure, Figure 4.23(B). Predicted and measured data are often in agreement.

CHAPTER 5 CONCLUSIONS AND RECOMMENDATION

The compressive strength of concrete and simple reinforced concrete beams is examined in this work, along with the impacts of concrete surface temperature and fire exposure duration. Both variables are tested in the residual condition, or after air cooling. Even if exposed to a small-scale fire for an extended period of time, concrete or reinforced concrete will drastically lose its strength, according to the finite element analysis results. In our nation, it typically takes more than four hours or even a day to contain a fully formed fire (this is because there aren't enough resources, access points, or communication channels from the fire site to the disaster region). Department of Fire). For all these reasons, the need to structure the fire class classification and incorporate fire safety considerations into the design process grows along with the economics and interest in safe fire resistance testing. due to the extremely high severity of fire accidents involving structural elements.

In our nation, it typically takes more than four hours or even a day to contain a fully formed fire (this is because there aren't enough resources, access points, or communication channels from the fire site to the disaster region, according to the Department of Fire). For all these reasons, the need to structure the fire class classification and incorporate fire safety considerations into the design process grows along with the economics and interest in safe fire resistance testing. due to the extremely high severity of fire accidents involving structural elements.

5.1 CONCLUSION

The following inferences can be made in light of the temperature curve that was discovered and the beam's reaction to the load.

This study allows for the following conclusions to be drawn:

- i. The data makes it evident that the concrete protective layer's thickness has no bearing on the reduction in resistance performance when subjected to fire for one to two hours after cooling to room temperature. That is, the thickness of the protective layer does not significantly affect the seismic behavior of the concrete. Built-in resistance-reinforced concrete steel bars.

- ii. It makes sense that the maximum temperature has a significant impact on regulating the concrete's residual compressive strength. The remaining parameters exhibit nearly identical sensitivity.
- iii. The compressive strength at 700 °C will decrease to roughly 80% based on the relationship between the temperature and the remaining strength of the beam.
- iv. The compressive resistance of the cube will not be greatly affected by exposure to a fire at 200°C. This is because even at 500°C, the compressive strength of concrete will decrease by about 45% to 60%.
- v. The steel reinforcement in the concrete is what determines the beam's residual performance because the temperature in the stress zone of the beam is less than 300°C, indicating that there is no visible damage to the covering concrete.
- vi. Samples to be cooled and afterwards analyzed for mechanical response following fire exposure. To confirm the precision and effectiveness of the suggested model, the outcomes of a more sophisticated finite element model are also given.
- vii. The cross-section of the RC beam is chosen using the 3D FE model to represent its typical thermal field, and the temperature distribution that is obtained is consistent with the findings reported in the literature.
- viii. When the concrete layer was 15 mm, the breaking load in reinforced concrete beams subjected to 700 degrees Celsius at 2, 3, and 4 hours was decreased by 40%, 64%, and 82%. As a result, kindly be aware that a particular lightning capability can also be decreased by the length of the fire.
- ix. From the results obtained the duration of time and intensity of fire are more sensitive compared to compressive strength and concrete cover.

5.2 RECOMMENDATION

Moreover, future research on the fire behavior of reinforced concrete beams under various application fire curves and boundary conditions can make use of the finite element model created and validated in this study. The deflection, maximum load capacity, and fire resistance time of structural elements can also be rather accurately predicted using finite element modeling. The modeling of shear stirrups in structural members can be disregarded for thermodynamic analysis.

The fire behavior of concrete has not been well studied, and more research is needed in practically every area of this science. The way that different concrete materials react to heat is essentially complicated. For instance, the way that concrete's physical qualities deteriorate depends a lot on the specifics of the concrete mix, such as the water content and other environmental factors that affect the fire's maximum temperature and duration. A careful investigation into the effects of various heating conditions on concrete is important.

This study shows that a fire accident in a service structure can cause a total collapse, which can lead to casualties. Therefore, due to the reasons mentioned and not mentioned, there must be construction rules and regulations to establish the factor of safety or allowable value of the fire resistance class of structural elements, as well as the method of determining the class of fire resistance of the existing structure.

The The effect of fire exposure to high temperatures The university built a fire testing compartment to test simultaneous loading during firing.

To assess the following, further research is recommended. These include:

- Effects of the use of cement types and collective types with different resistance to fire
- Effects of water on the ratio of cement or moisture content in the resilience of the structural element to fire
- Fire test concentration and distribution for several enriched loads
- Fire testing and suppression of specimens for different support conditions
- In the case of concrete spalling with contribution fires at different levels and durations, Effects of high concrete strength at different levels and durations

Furthermore, it is advised to test for failure load concurrently with fire resistance in order to obtain more accurate fire test findings on structural member fire resistance.

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Annex A

1) Cross-section property

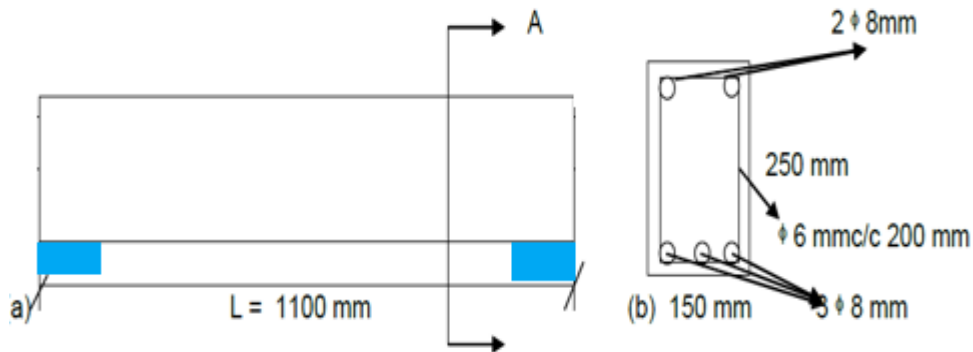


Figure A- 1 Sample beam cross-section

2) Density Concrete

EC2 Part 1-2 section 3.3.2(3),

The formula $\rho(\theta) = \rho(20^\circ\text{C})$ can be used for any temperature between 20°C and 115°C .

$$20^\circ\text{C} = \rho(\theta) = \rho \cdot (1 - 0.02(\theta - 115)/85) \text{ for } 115^\circ\text{C} \leq c \leq 200[^\circ\text{C}]$$

$$20^\circ\text{C} = \rho(\theta) = \rho \cdot (0.97 - 8.03(\theta - 200)/200) \text{ for } 200^\circ\text{C} \leq c \leq 400[^\circ\text{C}]$$

$$20^\circ\text{C} = \rho(\theta) = \rho \cdot (0.95 - 0.07(\theta - 400)/800) \text{ for } 400^\circ\text{C} \leq c \leq 1200[^\circ\text{C}]$$

where $\rho(20^\circ\text{C}) = 2300\text{kg/m}^3$ and c is the concrete temperature [$^\circ\text{C}$].

Table A- 1 Density of concrete at the level of temperature

Temperature	Density
20	2300
100	2308.117647
200	2254
300	2205.125
400	2185
500	2164.875
600	2144.75
700	2124.625

3) The density of steel bars

The mean value of density may be assumed to be 78500 kg/m^3

Table A- 2 Elastic modulus of steel reinforcement and concrete at a given temperature

C-25		
E(Pa)	V	Temp
30000000000	0.2	20
30000000000	0.2	100
28500000000	0.2	200
25500000000	0.2	300
22500000000	0.2	400
18000000000	0.2	500
13500000000	0.2	600
9000000000	0.2	700

c-30		
E(Pa)	V	Temp
31000000000	0.2	20
31000000000	0.2	100
29450000000	0.2	200
26350000000	0.2	300
23250000000	0.2	400
18600000000	0.2	500
13950000000	0.2	600
9300000000	0.2	700

steel s-500		
Ect(cold)	V	temp
2.1E+11	0.29	20
2.1E+11	0.29	100
1.8E+11	0.29	200
1.5E+11	0.29	300
1.2E+11	0.29	400
8.4E+10	0.29	500
5E+10	0.29	600
1.7E+10	0.29	700

Table A- 3 Heat Specification and Conductivity of S-500mpa Reinforcement Steel Bars

Specific Heat (J/Kg/K)	TEMP	Conductivity	Temo
440	0	53.3	0
440	20	53.3	20
488	100	50.7	100
530	200	47.3	200
565	300	44	300
606	400	40.7	400
667	500	37.4	500
760	600	34	600
1008	700	30.7	700

a) Specific heat Concrete EC2 Part 1-2 section 3.3.2(1),

$$C_p(\theta_c) = 900 \text{ (J/kg K)} \quad \text{for } 20[^\circ\text{C}] \leq \theta_c < 100[^\circ\text{C}]$$

$$C_p(\theta_c) = 900 + (\theta_c - 100) \text{ (J/kg K)} \quad \text{for } 100[^\circ\text{C}] \leq \theta_c < 200[^\circ\text{C}]$$

$$C_p(\theta_c) = 1000 + (\theta_c - 200)/2 \text{ (J/kg K)} \quad \text{for } 200[^\circ\text{C}] \leq \theta_c < 400[^\circ\text{C}]$$

$$C_p(\theta_c) = 1100 \text{ (J/kg K)} \quad \text{for } 400[^\circ\text{C}] \leq \theta_c < 1200[^\circ\text{C}]$$

Table A- 4 Specific heat Concrete EC2 Part 1-2 section 3.3.2(1),

Temperature	Specific Heat U=0%	Specific Heat U=1.5%	Specific Heat U=3%
TO	Cp(U=0%)	Cp(u=1.5%)	Cp(3%)
0	900	900	900
20	900	900	900
100	900	1470	2020
200	1000	1470	2020
300	1100	1100	1100
400	1100	1100	1100
500	1100	1100	1100
600	1100	1100	1100
700	1100	1100	1100

6) Thermal conductivity concrete

EC2 $\lambda_c = 2 - 0.2451 (\theta_c/100) + 0.0107(\theta_c/100)^2$ [W/mK] for $20^\circ\text{C} \leq \theta_c \leq 1200^\circ\text{C}$
 Part 2

lower limit

$$\lambda_c = 1.36 - 0.136 (\theta_c/100) + 0.0057(\theta_c/100)^2$$
 [W/mK] for $20^\circ\text{C} \leq \theta_c \leq 1200^\circ\text{C}$

a) Thermal conductivity concrete EC2 Part 2 Upper Limit $\lambda_c = 2 - 0.2451 (\theta_c/100) + 0.0107(\theta_c/100)^2$ [W/mK] for $20^\circ\text{C} \leq \theta_c \leq 1200^\circ\text{C}$

lower limit $\lambda_c = 1.36 - 0.136 (\theta_c/100) + 0.0057(\theta_c/100)^2$ [W/mK] for $20^\circ\text{C} \leq \theta_c \leq 1200^\circ\text{C}$

Table A- 5 Thermal conductivity concrete EC2 Part 2

temperature	upper limit	Lower limit
20	1.951408	1.333028
100	1.7656	1.2297
200	1.5526	1.1108
300	1.361	1.0033
400	1.1908	0.9072
500	1.042	0.8225
600	0.9146	0.7492
700	0.8086	0.6873

Table A- 6 Property Damage plasticity of concrete For fck-25 mpa

				0.4fcm	0.85fcm
Ecm	28960.41			10	21.25
cm	25	33			
eco	0.000345				
ec1	0.001899				
ECM	0.010468				
from	2.655857				
ecu1	0.010468				

Table A- 7 Compression damage the plasticity of concrete

strain	stress		inelastic	
0	0	0		
0.00017265	5	0.00017		
0.0003453	10	0	0	0
0.0004453	11.6632351	0.0001	0	0.06933
0.0005453	13.6633436	0.0002	0	0.11283
0.0006453	15.4647204	0.0003	0	0.14423
0.0007453	17.0786742	0.0004	0	0.16908
0.0008453	18.5156718	0.0005	0	0.19003
0.0009453	19.7854147	0.0006	0	0.20853
0.0010453	20.8969082	0.0007	0	0.22543
0.0011453	21.8585222	0.0008	0	0.24126
0.0012453	22.6780469	0.0009	0	0.25639
0.0013453	23.3627417	0.001	0	0.27107
0.0014453	23.9193796	0.0011	0	0.28548
0.0015453	24.3542877	0.0012	0	0.29976
0.0016453	24.6733828	0.0013	0	0.31402
0.0017453	24.8822045	0.0014	0	0.32834
0.0018453	24.9859445	0.0015	0	0.34279
0.0019453	24.989474	0.0016	0	0.35744
0.0020453	24.8973672	0.0017	0	0.37234
0.0021453	24.7139244	0.0018	0	0.38755
0.0022453	24.4431919	0.0019	0	0.40311
0.0023453	24.0889802	0.002	0.70627	0.41906
0.0024453	23.6548816	0.0021	0.71997	0.43544
0.0025453	23.1442848	0.0022	0.73354	0.45231
0.0026453	22.5603897	0.0023	0.74699	0.4697
0.0027453	21.9062202	0.0024	0.76036	0.48767
0.0028453	21.1846362	0.0025	0.77363	0.50624
0.0029453	20.3983443	0.0026	0.78684	0.52548
0.0030453	19.5499082	0.0027	0.79999	0.54543
0.0031453	18.641758	0.0028	0.81308	0.56615

0.0032453	17.6761986	0.0029	0.82613	0.5877
0.0033453	16.655418	0.003	0.83914	0.61012
0.0034453	15.5814941	0.0031	0.85211	0.6335

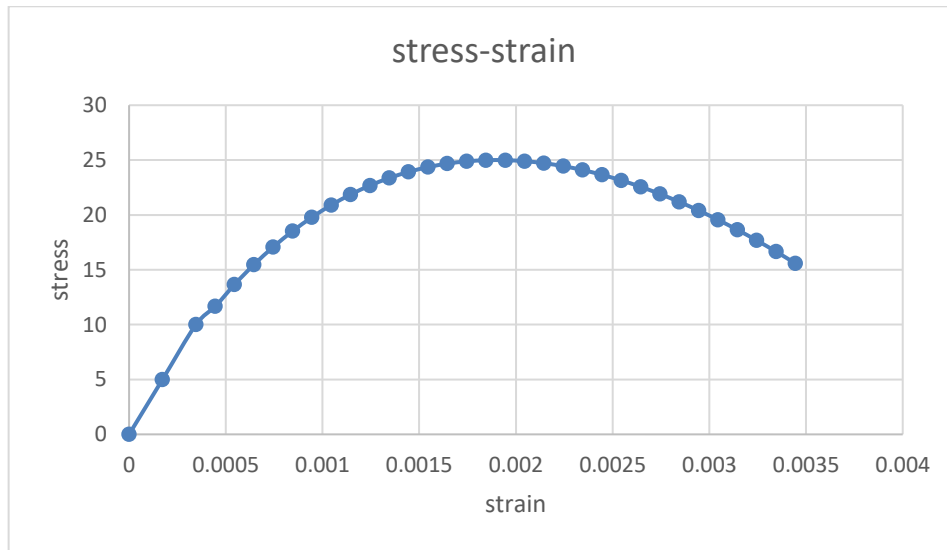


Figure A- 2 Stress-Strain Compression Damage The Plasticity Of Concrete

Table A- 8 Tensile damage plasticity of concrete

strain	stress		Inelastic
0	0		
9.17065E-05	2.655857493	0	0
0.00021	1.906676808	0.00012	0.64244
0.00031	1.631624149	0.00022	0.79485
0.00041	1.458986453	0.00032	0.86335
0.00061	1.244608802	0.00052	0.92343
0.00071	1.171281982	0.00062	0.9386
0.00081	1.111145149	0.00072	0.94929
0.00091	1.060591751	0.00082	0.95716
0.001052857	1.000499839	0.00096	0.9653
0.001174286	0.957756903	0.00108	0.97036
0.001295714	0.920791079	0.0012	0.97427
0.001417143	0.888381179	0.00133	0.97738
0.001538571	0.859642219	0.00145	0.9799
0.00166	0.83391458	0.00157	0.98197
0.001781429	0.810694892	0.00169	0.9837
0.001902857	0.789591252	0.00181	0.98517
0.002024286	0.770293225	0.00193	0.98642
0.00215	0.751950787	0.00206	0.98754
0.002272597	0.735454483	0.00218	0.98849
0.002395195	0.720159121	0.0023	0.98932

0.002517792	0.705922213	0.00243	0.99005
0.00264039	0.692624095	0.00255	0.9907
0.002762987	0.680163409	0.00267	0.99128
0.002885584	0.668453634	0.00279	0.99181
0.003008182	0.657420389	0.00292	0.99228
0.003130779	0.646999321	0.00304	0.9927
0.003253377	0.637134417	0.00316	0.99309
0.003375974	0.627776658	0.00328	0.99344
0.003498571	0.61888293	0.00341	0.99377

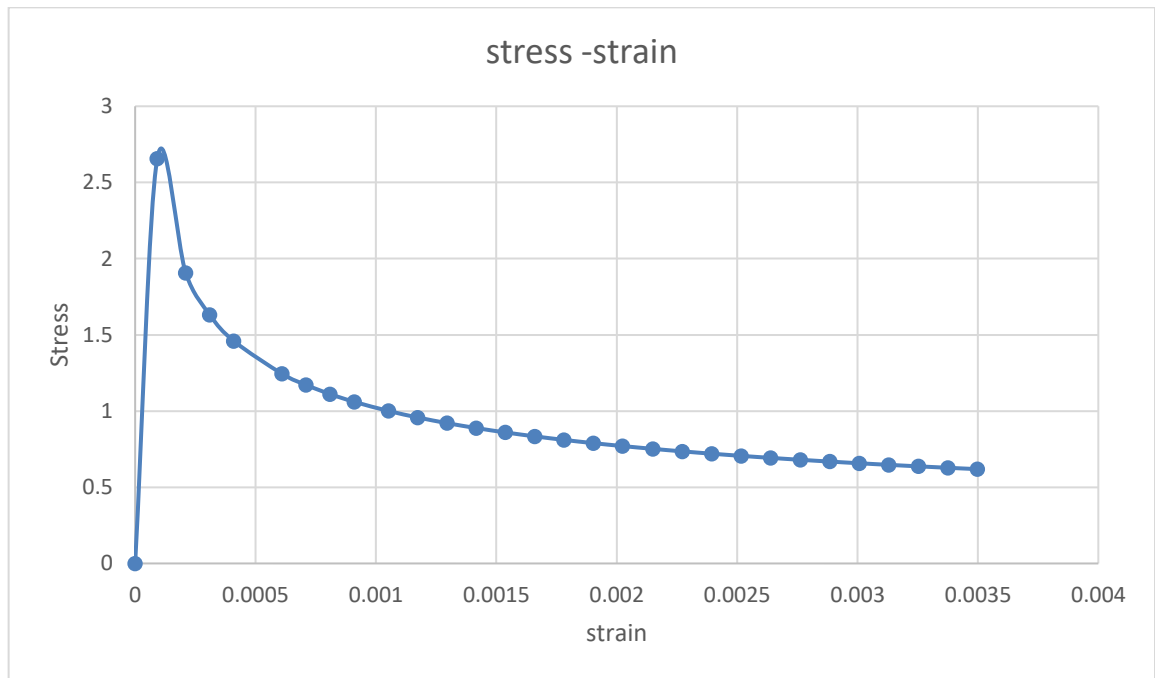


Figure A- 3 Stress-Strain Tensile damage plasticity of concrete

Table A- 9 Reinforcing Steel property

Es	200000
V	0.3
0.001Es	2000
Fy	500
Fu	590

strain	stress
0	500
0.045	590

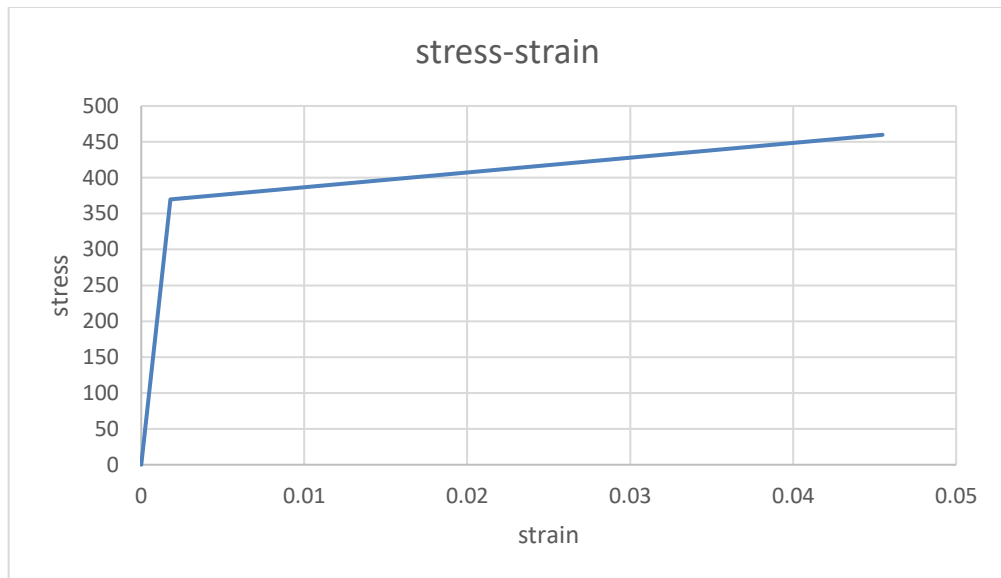


Figure A- 4 Stress-Strain Tensile Damage Plasticity Of Steel

Annex B

FEM Out put (ABAQUS)

Table B- 1 Result of compressive Strength 25Mpa of concrete and Relative strength after Burning

GROUP	Duration (hrs.)	Concrete Compressive Strength(Mpa)	Temperature (°C)	Concrete Compressive Strength after burning (Mpa)	Relative strength(% age)
GROUP 1*	0	25	20	25	100
Group 1	2	25	100	24.56	92.24
			200	23.89	95.56
			300	23.01	92.04
			400	21.25	85
			500	15.23	60.92
			600	14.09	56.36
			700	8.99	35.96
Group 2	3	25	100	24.05	96.2
			200	23.39	93.56
			300	22.29	89.16
			400	18.8	75.2
			500	13.26	53.04
			600	11.25	45
			700	6.25	25
Group 3	4	25	100	23.88	95.52
			200	22.15	88.6
			300	20.34	81.36
			400	16.2	64.8
			500	11.85	47.4
			600	9.23	36.92
			700	4.07	20.28

Table B- 2 Result of compressive Strength 30Mpa of concrete and Relative strength after Burning

GROUP	Duration (hrs.)	Concrete Compressive Strength(Mpa)	Temperature (°C)	Concrete Compressive Strength after burning (Mpa)	Relative strength(% age)
Group 1*	0	30	20	30	100
			100	29.36	97.87

Group 1	2	30	200	28.53	95.10
			300	27.86	92.87
			400	25.67	85.57
			500	16.15	53.83
			600	14.25	47.50
			700	11.23	37.43
Group 2	3	30	100	29.16	97.21
			200	28.08	94.59
			300	26.04	86.81
			400	22.35	74.50
			500	15.24	50.80
			600	11.98	39.93
			700	7.84	26.13
Group 3	4	30	100	28.92	96.40
			200	26.58	88.60
			300	24.35	81.18
			400	19.58	65.27
			500	13.56	45.20
			600	10.57	35.23
			700	5.64	18.80

Table B- 3 Failure load after burning and Relative strength (% age) result of the reinforced beam after burning with concrete cover CC-15mm and FCK-25Mpa

Group	Concrete Cover (mm)	Concrete Compressive Strength(Mpa)	Duration (hrs.)	Temperature (°C)	Failure Load After Burning (KN)	Relative strength (%)
Group 1*	15	25	0.00	20	110.49	100.00
Group 1	15	25	2	100	108.56	98.25
				200	106.84	96.70
				300	103.80	93.95
				400	98.74	89.37
				500	87.34	79.05
				600	78.34	70.90
				700	66.24	59.95
Group 2	15	25	3	100	106.84	96.70
				200	104.25	94.35
				300	98.90	89.51
				400	92.02	83.28
				500	73.36	66.40
				600	65.23	59.04
				700	50.68	45.87

Group 3	15	25	4	100	101.34	91.72
				200	98.75	89.37
				300	93.94	85.02
				400	75.04	67.92
				500	60.35	54.62
				600	53.90	48.78
				700	32.25	29.19

Table B- 4 Failure load after burning and Relative strength (% age) Result of reinforced beam after burning with concrete cover CC-25mm and FCK-25Mpa

Group	Concrete Cover (mm)	Concrete Compressive Strength(Mpa)	Duration (hrs.)	Temperature (°C)	Failure Load After Burning (KN)	Relative strength (%)
Group 1*	25	25	0.00	20	106.26	100.00
Group 1	25	25	2	100	104.80	98.63
				200	97.41	91.67
				300	93.10	87.62
				400	89.30	84.04
				500	76.30	71.81
				600	72.10	67.85
				700	58.50	55.05
Group 2	25	25	3	100	102.64	96.59
				200	97.57	91.82
				300	94.40	88.84
				400	87.62	82.46
				500	68.50	64.46
				600	52.32	49.24
				700	43.74	41.16
Group 3	25	25	4	100	98.20	92.41
				200	91.20	85.83
				300	83.00	78.11
				400	68.25	64.23
				500	51.54	48.50
				600	42.40	39.90
				700	24.68	23.23

Table B- 5 Failure load after burning and Relative strength (% age) result of the reinforced beam after burning with concrete cover cc-15mm and FCK-30Mpa.

Group	Concrete Cover (mm)	Concrete Compressive Strength(Mpa)	Duration (hrs.)	Temperature (°C)	Failure Load After Burning (KN)	Relative strength (%)
Group 1*	15	30	0.00	20	123.65	100.00
Group 1	15	30	2	100	121.21	98.03
				200	119.87	96.94
				300	116.25	94.02
				400	112.98	91.37
				500	95.64	77.35
				600	86.69	70.11
				700	75.58	61.13
Group 2	15	30	3	100	118.57	95.89
				200	114.69	92.75
				300	108.47	87.72
				400	102.97	83.28
				500	92.74	75.00
				600	84.46	68.30
				700	70.88	57.32
Group 3	15	30	4	100	114.54	92.63
				200	107.54	86.97
				300	97.54	78.88
				400	82.14	66.43
				500	71.58	57.89
				600	63.09	51.02
				700	51.24	41.44

Table B- 6 Failure load after burning and Relative strength (% age) result of the reinforced beam after burning with concrete cover CC-25mm and FCK-30Mpa.

Group	Concrete Cover (mm)	Concrete Compressive Strength(Mpa)	Duration (hrs.)	Temperature (°C)	Failure Load After Burning (KN)	Relative strength (%)
Group 1*	25	30	0.00	20	113.08	100.00
Group 1	25	30	2	100	110.47	97.69
				200	106.67	94.33
				300	99.95	88.39
				400	93.07	82.31

				500	80.02	70.76
				600	73.69	65.17
				700	61.04	53.98
Group 2	25	30	3	100	108.95	96.35
				200	98.65	87.24
				300	93.23	82.45
				400	81.23	71.83
				500	74.25	65.66
				600	68.32	60.42
				700	54.25	47.97
Group 3	25	30	4	100	106.87	94.51
				200	98.50	87.11
				300	81.53	72.10
				400	74.28	65.68
				500	64.12	56.70
				600	55.78	49.33
				700	48.48	42.87