

**DEPARTMENT OF CIVIL ENGINEERING
AHMADU BELLO UNIVERSITY, ZARIA**

**BEHAVIOUR OF CONCRETE SUBJECTED TO VARIOUS
ELEVATED TEMPERATURES**

BY

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PhD/ENG/05541/2008-2009**

February, 2014

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**A DISERTATION SUBMITTED TO THE POST GRADUATE SCHOOL AHMADU
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IN CIVIL ENGINEERING**

**IN THE DEPARTMENT OF CIVIL ENGINEERING
FACULTY OF ENGINEERING AHMADU BELLEO UNIVERSITY, ZARIA**

FEBRUARY, 2014

DECLARATION

I, Dahiru Salisu, hereby declare that this dissertation work entitled **“BEHAVIOUR OF CONCRETE SUBJECTED TO VARIOUS ELEVATED TEMPERATURES** has been designed and implemented by me in the Department of Civil Engineering under the supervision of Dr Y.D. Amartey, Prof. S.P. Ejeh and Dr. I. Abubakar all in the Department of Civil Engineering, Ahmadu Bello University, Zaria. It is therefore, a record of a research work I carried out myself and has not been accepted in any institution for an award of higher degree. The sources of information have been duly acknowledged in the text by means of references, and quotations there from have been distinguished by means of quotation marks.

Salisu Dahiru

February, 2014

CERTIFICATION

This Dissertation titled “**Behaviour of Concrete Subjected to Various Elevated Temperatures**” by Dahiru Salisu, meets the regulations governing the award of the Doctor of Philosophy (PhD) in Civil Engineering of Ahmadu Bello University, Zaria and is approved for its contribution to knowledge and literary presentation.

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DEDICATION

This Dissertation is fully dedicated to my late parents (Alhaji Dahiru Aliyu Yau-yau & Hajiya Zulaiha Yusuf) may Allah forgive them. It is also dedicated to my two wives and children, with whom I got all the inspiration and strength.

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Abstract

In this research, the influence of aggregates sizes, cover to reinforcement thickness, concrete age, exposure period on the mechanical properties of normal concrete (NC) subjected to different elevated temperatures were investigated. Two types of concrete (combined aggregates concrete and single aggregate size concrete) were used to determine the residual compressive strength, while only combined aggregate concrete was used for flexural strength of reinforced concrete prisms. Fire resistance of the concrete cubes and concrete prisms specimens in terms of residual compressive and bending strength was studied. Five different temperatures level of 100⁰C, 300⁰C, 500⁰C, 700⁰C and 900⁰C were used and they were maintained constant for four different exposure periods of 30mins, 60mins, 90mins and 120mins. The residual compressive strength as percentage of the strength of the unheated specimens ranged between (95-120%) at 100⁰C, (70 -101%) at 300⁰C, (45-60%) at 500⁰C and (0-10%) between (700⁰C -900⁰C).

Exposure time above 1hour had significant effect on the residual compressive strength of concrete; however, the effect was so pronounced at early age concrete and single and bigger aggregates size concrete (20mm) when the temperature increased. Generally, the flexural strength was found to be more sensitive to high temperature exposure than compressive strength. The residual flexural strength was in the range of (85-90%), (50-75%) and (25-40%) at 100⁰, 300⁰C and 600⁰C respectively. The results clearly showed the influence of concrete cover to reinforcement on the residual deflection and tensile strength of concrete prisms specimens.

The respective deflection recorded for the three different concrete cover of 20, 25 and 30mm at 100⁰C and heated for 60min are (15, 20 and 23mm). The deflection was (20, 22 and 29mm) at 300⁰C when heated for the period of 60min, and the deflection was (31, 34 and 40mm) at 600⁰C when heated for the period of 120min respectively.

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CHAPTER ONE

INTRODUCTION

1.1 Introduction

Concrete, a leading construction material in civil engineering is sometimes exposed to elevated temperatures due to natural hazard Vodak *et al.*, (2004). Subjecting concrete to high temperatures leads to transformations and reactions that cause the progressive breakdown of cement gel structure and consequent loss in load-bearing capacity Khoury, (1992); Earline. *et al.*, 1972; Hanson,(1990); Handoo *et al.*, (2002)

Several processes have been identified for the deterioration of concrete due to high temperature. These include decomposition of the calcium hydroxide with time and water, expansion of lime on re-hydration, destruction of gel structure, phase transformation in some types of aggregate, and development of micro-cracks due to thermal incompatibility between cement paste matrix and aggregate phase Mohammad et al, (2009). Chandramouli, (2011)

Concrete as composite material consisting of steel and concrete, its strength (compressive and bending strengths) decreases as the temperature increases. Heating considerably changes the strength and physical properties concretes structural components such beams, column, wall and slabs. Gruz, (1966).

1.2 What is a fire? fire resistance & fire resistance requirement

Fire itself is a physical and chemical phenomenon that is strongly interactive by nature. Zhaohui et, al, (2007). M. Petru,(2008)

- ❖ Fire resistance is the ability of a structural element to maintain its load-bearing functions under standard fire conditions such as “ ISO 834:, ASTM E 119 (2001)

- ❖ A structural member is said to have possessed fire resistance requirements if it has the following

1. Integrity
2. Insulation
3. Structural adequacy

1.3 Background of the Research

The mechanical properties such as compressive strength, bending strength and modulus of elasticity of concrete are considerably reduced during fire events. This may result to considerable damage and undesirable structural performances. Sanjayan, (1993); Omer. (2009).

One of the main structural elements within concrete structures is the beams. As the beams form the support for other load-bearing elements like slabs, their collapse during fire can be detrimental to the stability of the entire structure. Cand-Ing, (2004), and Chandramouli, (2011)

The objective of fire safety is to protect life and property, and to ensure the integrity of the structure is maintained. Accidental Fires can occur at any given moment in buildings, and its negative consequences are always enormous on life and property. The safety of occupants and maintaining the integrity of the structure are of major importance. Building codes prescribe detailed measures for the fire safety of structural members because when other means for putting off a fire fail, such as fire suppression system ,structural integrity is the last line of defense. Sanjayan et, al (2012)

In the past decades, the fire performance of concrete has often been taken for granted considering its non-combustible nature and its ability to function as a thermal barrier, preventing heat and fire from spreading. Design criteria have been based on the results of testing to ‘standard’ fire

exposures (BS 8110, 1997; Eurocode 2, 2002) typically expressed in terms of required reinforcement cover or axis distance.

1.4: Statement of Problem

There are several cases of fire accident engulfing many life and properties and destroying many important structures sometime even razing them down completely. The Codes provide 1 hour or 2 hours maximum for fire fighters to arrive and put it off. In cases where fire fighters are not coming on time, the need to know the damage done and how it can be remedied cannot be over-emphasized.

1.5: Justification of Research

At the Department of Civil Engineering, Ahmadu Bello University, Zaria, Considerable research has been extensively conducted on the behaviour of Concrete and concrete structures as a whole under different aggressive environmental conditions. However, no similar research has been undertaken in to the behaviour of concrete subjected to various elevated Temperatures.

1.6: SCOPE AND LIMITATION OF THE WORK:

The scope of the present work is focused on small-scale non- standard -fire- testing with particular emphasis to provide an experimental database on the response of concrete structural components heated in electrical furnaces rather than real fire testing.

1.7. RESEARCH METHODOLOGY

This research is purely a Laboratory work which entails, casting, curing and heating of concrete specimens, crushing and bending test of the heated specimens.

1.8 Aim and Objectives of the Study

1.8.1 AIM

The main aim of this research work is to experimentally investigate the effects of the following parameters on the response of concrete elements subjected to various elevated temperatures.

1. aggregates size,
2. exposure periods,
3. concrete aging and
4. concrete cover to reinforcement

1.8.2: OBJECTIVES

The objectives of this research work are to:

- a. Determine the physical properties of Ordinary Portland Cement commonly used in Construction Industry in Nigeria (Dangwote Cement)
- b. Determine the physical properties of crushed aggregates.
- c. Produce the test specimens which include 150mm standard cubes and square prisms of 150x150 x750mm respectively with different curing age
- d. Subjecting the test specimens to varying elevated temperatures and exposure time
- e. Conduct crushing and bending tests on the heated specimens in order to determine both residual compressive and bending strengths of the heated specimens.
- f. Identify critical exposure temperature and exposure time beyond which failure is inevitable.
- g. Compare the test results with standards and specifications requirements.
- h. Putting forward appropriate recommendations from the obtained result.

1.9. Research Outcome:

At the end of this research work, it has been observed that heating concrete cubes specimens to 100°C resulted in a noticeable increase in compressive strength amounting to an average of 5-15%. However, such increase has not been observed on flexural strength of concrete. Beyond 300°C, the temperature will begin to damage the elemental build up of the concrete structure.

CHAPTER TWO:

LITERATURE REVIEW

2.1 INTRODUCTION

Concrete, a leading construction material in civil engineering is sometimes exposed to elevated temperatures due to either natural fire or as in an industrial application where concrete structures are close to reactors and electrical furnaces (Vodak *et al.*, 2004). According to Khoury, (1992); Anonymous, (1972); Erline *et al.*, (1972); Hanson, (1990); Handoo *et al.*, (2002), and Abrams *et al.* (1968) subjecting concrete to high temperatures leads to transformations and reactions that cause progressive breakdown of cement gel structure and consequent loss in load-bearing capacity.

In addition, several factors have been identified for the deterioration of concrete due to high temperature. These include decomposition of the calcium hydroxide with time and water, expansion of lime on re-hydration, destruction of gel structure, phase transformation in some types of aggregate, and development of micro-cracks due to thermal incompatibility between cement paste matrix and aggregate phase Mohammad *et al.*, (2009), (Abrams, (1973), Cruz, 1966)

2.2.1 Physical and Chemical response of Concrete

High temperatures cause chemical and micro structural changes, such as water migration (diffusion, drying), increased dehydration, interfacial thermal incompatibility, and chemical decomposition of hardened cement paste and aggregates. According to Zhang *et al.*, (2000), Bresler, *et al.* (1985) and Lie *et al.* (1985) these changes cause decrease in both the strength and stiffness of concrete and lead to irrecoverable deformation of structural components. Furthermore, the effect of elevated temperatures at varying heating scenarios on the strength of

plain concrete has been investigated by many researchers such as Mohammedbhai, (1983); Khalaf and Devinny, (2004); Chan *et al.*, (1999); Min *et al.*, (2004); Poon *et al.*, (2001); Phan *et al.*, (2001); Xiao and Falkner, (2006); Mahdy *et al.*, (2002); Abramowicz and Kowalski, (2005); Jianzhuang and Konig, (2004); Phan and Carino, (2000), and finally Felix *et al.*, (2010)

According to Gruz, (1966), Chew, (1993) during fire event, concrete elements (beams, columns, slabs, walls and sometimes foundations) are exposed to different heating scenarios (temperatures). Heating considerably changes the strength, stiffness as well as the physical properties of both concrete and steel. During heating, the steel reinforcement loses their flexural strength as well as concrete in the compression zone loses its compressive strength Neville, (1995) - Smith, *et al.*, (1979), Smith, *et al.*, (1979), and Uddin, *et al.*, (1975).

Riley, (1991), Hammer, (1995), and Saad *et al.*, (1996), reported that at temperatures approximately above 430° C, concretes with siliceous aggregates show significant strength loss when compared to those with light-weight aggregates.

Husem (2006), Cruz, (1966) examined the variation in compressive and flexural strengths of ordinary and high-performance concretes exposed to high temperatures of 200, 400, 600, 800 and 1000°C and then cooled in air or water. The compressive and flexural strengths of these concrete specimens were compared with each other and with unheated specimens. From the results obtained, he concluded that (a) for ordinary and high-performance concrete exposed to high temperature, the flexural and compressive strengths decrease with the increase in temperature. Such decrease is greater when the specimens were cooled in water.

In the same vein, Malhotra (1956), Davis (1967), Abrams (1971), Faiyadh (1989), Khoury (1992) and Noumowe *et al.* (1994) had reported the effects of high temperature exposure on the properties of concrete. Bazant and Kaplan, (1996), Turkey *et al.* (2001) investigated the

micro-structure and strength of the concretes exposed to fire. The results showed that, at 600° C, concrete can lose half of its strength.

Edwards et,al (1986) studied the effect of high temperatures (up to 800°C) on compressive strength, flexural strength and volume changes of high strength concrete (HSC). He concluded that high strength concrete, (HSC) is more sensitive to high temperatures than normal strength concrete (NSC). The residual compressive strength recorded ranged between (90-106 %) at 100 °C, (72-103 %) at 300°C, (55-87 %) at 500°C and (22-66 %) between (600-800°C). He concluded that exposure time beyond one hour had a significant effect on the residual compressive strength of concrete;

The influence of high temperature on concrete consisting fireclay brick was investigated by Usmani et al; (2002). The results showed maximum reduction in compressive strength of 25% to 60% at 450 to 600⁰c. Gallic, et al, (2001), concluded that, the chemical composition and physical structures of the concrete significantly changed when exposed to high temperature. Micro-cracks were noticed on specimen's surface as the temperature exceeds 300⁰c. Recent work by Kumar et, al, (2006) revealed that, Calcium hydroxide [Ca(OH)₂], which is one of the most important compounds in the cement paste, dissociates at around 530⁰c resulting in shrinkage of concrete.

A research work conducted by Ahmed et,al, (1987) revealed that, Calcium –Silicate (CSH) gel, which is the strength giving compound of cement paste, decomposes further above 600⁰c and the concrete crumbled at around 800⁰c. Umran (2002), posits that, the fire exposure effect on some mechanical properties of concrete. Three temperature levels of (400,500, and 700) °C were chosen with four different exposure duration of 0.5, 1.0, 1.5, and 2.0 hours without any imposed loads during heating. He found that the residual compressive strength ranged between (70-85%) at 400 °C, (59-78 %) at 500 °C and (43-62 %) at 700 °C. Kalkan (2010) further found that, the

flexural strength was found to be more sensitive to fire flame exposure than the compressive strength.

Fahmi (1986) conducted a research work on mechanical properties of concrete made with fine and coarse aggregate of different materials including quartz, pyrogoskit, gypsum and kaolinite, the results revealed that, heating concrete to a temperature range of 450⁰c to 600⁰c caused decrease in the compressive strength of about 36% to 54% respectively. Petru, (2008) reported a similar result.

In addition, Abram (1918) reported that, a number of physical and chemical changes such as spalling and dehydration of calcium hydroxide occurs in concrete subjected to fire which significantly weaken the concrete structure. According to Ian et, al, (2005); Khoury, (2004), when the temperature reaches 400⁰c, the calcium hydroxide in the cement will begin to dehydrate, generating more water vapor which consequently brings significant reduction in the physical strength of the concrete. Other change may occur in the aggregates at higher temperature. For example, quartz- based aggregates increase in volume, due to mineral transformation at about 800⁰c. (A.C.I 216-1-07& TMS-216-07, 2007)

Furthermore, Allen and Desai, (1992) used three different aggregate: a light weight shale, a fireclay brick and a pure limestone in the strength of concrete subjected to high temperature. The concrete consisting fireclay brick showed the best mechanical properties while limestone deteriorated most considerably with maximum reduction in compressive strength of 25 to 60%.

2.2.2 Protection against Fire

There are two distinct aspects of fire protection: life safety and property protection. Although providing for one aspect generally results in some protection for the other, the two goals are not mutually inclusive. A program that provides for prompt notification and evacuation of occupants

meets the objectives for life safety, but provides no protection for property. Conversely, it is possible that adequate property protection might not be sufficient for protection of life. Absolute safety from fire is not attainable. It is not possible to eliminate all combustible materials or all potential ignition sources. Thus, in most cases, an adequate fire protection plan must assume that unwanted fire will occur despite the best efforts to prevent them. Means must be provided to minimize the losses caused by the fire that do occur. Underberg, Y. (1986).

The first obligation of designers is to meet legal requirements while providing the facilities required by the client. In particular, the requirements of the applicable building code must be met. The building code will contain fire safety requirements, or it will specify some recognized standard by reference most favorable insurance rate, if for no other reason.

Although fire in buildings can be avoided, they nevertheless occur. Some of the reasons for this are human error, faulty electrical equipment, poor maintenance of heating equipment, and natural causes, such as lightning. Consequently, buildings should be designed to minimize the probability of a fire and to protect life and limit property damage if a fire should occur. The minimum steps that should be taken for the purpose are as follows: ASTM. (2001). Gustefero et, al, (1986) and Harmathy, et, al (1987)

1. Limit potential fire loads, with respect to both combustibility and ability to generate smoke and toxic gases.
2. Provide means for prompt detection of fire, with warnings to occupants who may be affected and notification of the presence of fire to fire fighters.
3. Communication of instructions to occupants as to procedures to adapt for safety, such as to staying in place, proceeding to a designated refuge area, or evacuating the building.

4. Provide means for early extinguishment of any fire that may occur, primarily by automatic sprinklers but also by trained fire fighters. ASTM- ASTM E119-11a (2001).
5. Make available also for fire fighting an adequate water supply, appropriate chemicals, adequate-size piping, conveniently located valves on the piping, hoses, pumps, and other equipment necessary.
6. Prevent spread of fire from building to building, either through adequate separation or by enclosure of the building with incombustible materials.
7. Partition the interior of the building with fire barriers, or divisions, to confine a fire to a limited space.
8. Enclose with protective materials structural components that may be damaged by fire (fire-proofing materials).
9. Provide refuge areas for occupants and safe evacuation routes to outdoors.
10. Provide means for removal of heat and smoke from the building as rapidly as possible without exposing occupants to these hazards, with the air-conditioning. ASTM- ASTM E119-11a (2011).

2.2.3 Fire-Protection Standards

The standards most widely adopted are those published by the National Fire Protection Association (NFPA), Battery march Park, Quincy, MA 02269. The NFPA “National Fire Codes” comprise several volumes containing numerous standards, updated annually. (These are also available separately.) The standards are supplemented by the NFPA “Fire Protection Handbook,” which contains comprehensive and detailed discussion of fire problems and much valuable statistical and engineering data.

In addition, buildings must comply with provisions of the Americans Disability Act (ADA). (See Department of Justice final rules, Federal Register, 28 CFR Part 36, July 26, 1991; American National Standards Institute “Accessibility Standard,” ANSI A117.1; “ADA Compliance Guidebook,” Building Owners and Managers Association International, 1201 New York Ave., Washington, D.C. 20005)

2.2.4 Effects of Aggregate Type on Residual Compressive Strength of Concrete.

Abdellalim et al, (2009), Yuzer, et, al (2004) indicated in their finding that, aggregate type has a minor effect on the concrete resistance to fire. However, the result showed that, dolomite aggregates provided the highest resistance to fire while natural aggregates gave the least resistance. On the other hand, aggregates type plays an important role on the concrete spalling under fire conditions.

Zoldners, (1960.) concluded that the change in concrete properties due to high temperature depends on the type of coarse aggregate used. Carbonate (limestone and dolomite). Siliceous (silica, granite and sandstone) and Lightweight aggregates (shale, slate, or clay).

Sullivan and Poacher, (1971) reported that, the quality of concrete greatly depends on the sound inherent properties of concrete aggregates, such as chemical and physical properties.

2.2.5 Bonding between aggregates and Cement.

Abdellalim, et al, (2009) advised that, bond between aggregates and cement matrix depends on the mechanical keying of the hydration products of cement matrix with the rough aggregate surface. It also relies on the physic-chemical bond between the hydrating cement paste and the aggregates surface as a result of the chemical reaction at the interfacial zone. It has also being deduced that, aggregates expand during heating while cement paste shrinks due to loss of water.

This phenomenon leads to the breaking the bond between the cement matrix and the coarse aggregates.

2.2.6 Chemical Composition of Aggregates

According to Blank, and Kennedy, (1955), the chemical phenomenon of concrete aggregates is divided in to two general groups: (a) those which produce effects such as surface popouts and surface staining, etc.; resulting from oxidation, hydration, and carbonation reactions of certain unstable mineral particle (b) those which are responsible for volume changes which disrupt the integrity of the concrete resulting from so-called alkali-silica and alkali-carbonate reactions

2.2.7 Thermal Capacity of Aggregates.

Tremper et, al (1958), Arioiz (2009), Anderberg, et,al (1977), Alarco et,al, (2005) reported that, aggregate particles have three thermal properties, a coefficient of thermal expansion, a specific heat value, and thermal conductivity. The specific heat and conductivity of concrete greatly affect the durability of concrete in areas subject to significant temperature rise. The differential thermal expansion and stiffness characteristics between the cement paste and aggregate contribute to the development of micro-cracking at the interface.

2.2.8 Physical Effect of Aggregates

Khoury, (1992), Chandramouli et al, (2011) studied the physical properties of aggregates and found that, it may profoundly affect the quality of a concrete include water absorption; particle shape and surface texture; volume change with wetting and drying; strength, elasticity, hardness, aggregates type, shape and size. The aggregate shape also influences concrete permeability. The hardness of aggregate particles depends only on the hardness of their constituent minerals, not upon the firmness with which the constituents are bound together. Aggregates must conform to certain standards for optimum engineering use. They must be clean, hard, strong, durable, and

free of absorbed chemicals, coating of clay and other fine materials in amounts that could affect hydration and bond with the cement paste. Aggregate particles that are friable or capable of being split are undesirable. The essential requirement of an aggregate for concrete is that it remains stable within the concrete and in the particular environment throughout the design life of the concrete.

The use of elongated and flaky aggregates significantly increases the permeability by trapping sizeable mobile bleed water and air bubbles under their flat surfaces, thereby producing under-aggregate fissures and zones. This results in a weak transition zone in which micro-cracks can easily form. These micro-cracks are influential contributing factor in enhancing the permeability of hardened concrete. The proportioning of coarse and fine aggregates is also important for the production of a workable and durable concrete. The aggregate proportioning for this purpose consists of fixing the optimum volume fraction of sand in the total aggregate content. Studies conducted by Tremper, et al. (1958) revealed that both ‘too high’ and ‘too low’ sand reduce the workability of concrete.

The aggregate characteristics play a major role in determining the properties of hardened concrete. Aggregates occupy about 70% to 80% of concrete by volume therefore; it's not surprising that its quality is of considerable importance. Aggregates were originally viewed as inert materials. In fact, aggregate is not truly inert and its physical, thermal and sometimes chemical properties influence the performance of concrete, as aggregates with undesirable properties cannot produce strong concrete. Khoury and Underberg (1986),

Aggregates are cheaper than cement and it is economical to put into the mix as much of the aggregates and little of cement as possible. But economy is not the only reason for using aggregates; it confers considerable technical advantages to concrete, which has a higher volume

stability and better durability than hydrated cement paste. Rininger,(2006). According to Liu et al, (2010). Aggregate characteristics can be divided into two groups: physical features (Particle size and shape and texture) and quality features (strength, density, porosity, hardness, and elastic moduli, chemical and mineral composition.

Rininger, et, al (2009), Anonymous (1972) revealed that aggregate texture, structure, and composition are responsible for the development of a characteristic strength and elasticity. Most normal construction steels have well-defined yield strength at normal temperatures. However, this yield point disappears at elevated temperatures (Buchanan, 2001). The reduction of the ultimate and yield strengths of steel. There is a significant amount of scatter shown with various curves recommended by various researchers. The vertically hatched and the horizontally hatched area represent the ultimate strength and yield strength of the steel.

2. 2.9 Fire Loads and Resistance Ratings

The nature and potential magnitude of fire in a building are directly related to the amount and physical arrangement of combustibles present, as contents of the building or as materials used in its construction. Because of this, all codes classify buildings by occupancy and construction, because these features are related to the amount of combustibles. ASTM E119-00a, (2000). The total amount of combustibles is called the fire load of the building Both, et al;. (2003).

2.2.10 Spalling

According to Sanjayan and Stocks (2007), this is the phenomenon involving the disintegration of part of concrete surface, due to the breakdown in surface tensile strength, caused by mechanical forces generated within the element due to strong heating. Aly, et, al (2001) reported that as the temperature increases, the induced stresses increase gradually to a limit that exceeds the concrete tensile strength and consequently small scales and sometime portion fallout from the concrete

surface either progressively or explosively depending on the degree of temperature and concrete type and grade

Blank and Kennedy (1955) revealed that, whenever concrete element is subjected to high temperature, free and combined water inside the concrete starts to evaporate and form pore pressure builds up and induces stresses on the internal structure of the concrete elements. As a result of the above negative effects, bonding between concrete and reinforcing bars will be greatly reduced. According to Anderberg,(1976) the temperature at which explosive spalling is most likely to occur depends upon the rate at which pore pressures and thermal stresses increase and material strength decreases. At this point, the bursting and resistance stresses moves in favour of the bursting stresses.

Meyer Ottens, (1972) tested concrete members from three different sides and concluded that spalling was less likely when concrete was heated from one side and occurred only at higher heating rates. Connelly (1995) confirmed the conclusions of Meyer- ottens, (1972) that, concrete heated from one side is less likely to experience explosive spalling in fire than concrete heated both sides. Bailey et al, (2006), Ian et, al (2004) found that high strength concrete slabs heated to (ISO 834:1987) standard fire did not show any surface distress. This may have been due to heating on one side of the flat surface and the relatively small size of the slab,

2.2.11 Explosive Spalling of Concrete

Structural fire design of modern reinforced and pre-stressed concrete structures relies on the assumption that the concrete will not spall during fire. This assumption is based largely on data from large-scale standard fire tests of concrete elements tested in isolation in furnaces during the past 60 to 70 years. However, there is legitimate concern (Kelly and Purkiss, 2008) that modern concrete structures, which incorporate concrete mixes with considerably higher concrete strengths, are more susceptible to spalling than was historically the case. While preliminary

guidance on the means by which spalling can be addressed by designers is available in the structural Euro codes (CEN, 2004), research is badly needed to understand the respective roles of the various factors which are known to increase a concrete's propensity for spalling during fire (e.g., high strength, high stress, high moisture, low permeability, small amounts of bonded reinforcement, use of silica fume, rapid heating, etc.) Arup, (2005); Bailey and Khoury, (2011), such that defensible preventative actions can be taken for instance the requirement to add a certain amount of polypropylene fibers to the concrete mix. Interactions in real structures have the potential to significantly influence development of spalling in a fire, so large scale tests under natural fires are needed to truly understand propensity for, and consequences of, spalling in real buildings.

2.2.12 Factors Influencing Spalling and Cracking

BS 8110: Part 2:1985 states that “rapid rates of heating, large compressive stresses or high moisture contents can lead to spalling of concrete cover at elevated temperatures, particularly for thicknesses exceeding 40 mm to 50 mm.” From literature many factors have been identified in influencing explosive spalling in concrete. Their influence is summarized as follows: Meyer Ottens (1977) reported that, heating rate has a major influence on the occurrence of explosive spalling. However, when spalling does occur it does so within a consistent temperature range, regardless of heating rate. Khoury et,al (1985), Kodur, (2008). Khoury & Ander berg, (2000) revealed that, heating from two faces is more likely to encourage spalling than heating from one face only

Sullivan et al, (1971) presented experimental evidence which suggested that, influence of concrete age on its susceptibility to spalling has been subject of conflicting reports. The majority of reports, however, suggest that the probability of spalling reduces with age. However,

experience from full scale fire tests carried out by the Fire Research Station by Bullock indicates that concrete held in an indoor environment for over 20 years is capable of violent explosive spalling. It was reported that permeability is an important factor influencing the critical pressure level, because it affects the rate of vapour release. Harmathy (1965) suggested that spalling is unlikely to occur if the permeability of the concrete is greater than about $5 \times 10^{-11} \text{ cm}^2$.

Zhukov, (1975) observed an increased tendency of concretes with high moisture contents to spall. He produced a spalling envelope similar to that of Christiaanse which indicate that spalling will not occur for moisture contents below 4% by weight.

Copier, 1979) concluded that moisture content is the most important factor in the determination of the likelihood of spalling.

Connolly, R.J. (1995) suggests that increasing the aggregate size promoted explosive spalling. It was reported from literature that internal cracking has a dual and opposite effect upon explosive spalling. While micro cracks facilitate the escape of moisture during heating and thus relieve pore pressures, they also facilitate the process of spalling by providing a source for crack propagation. Malhotra, (1984) concluded that if the cover to reinforcement exceeds 40 mm for dense or 50 mm for lightweight aggregates concrete, there is a danger of concrete spalling

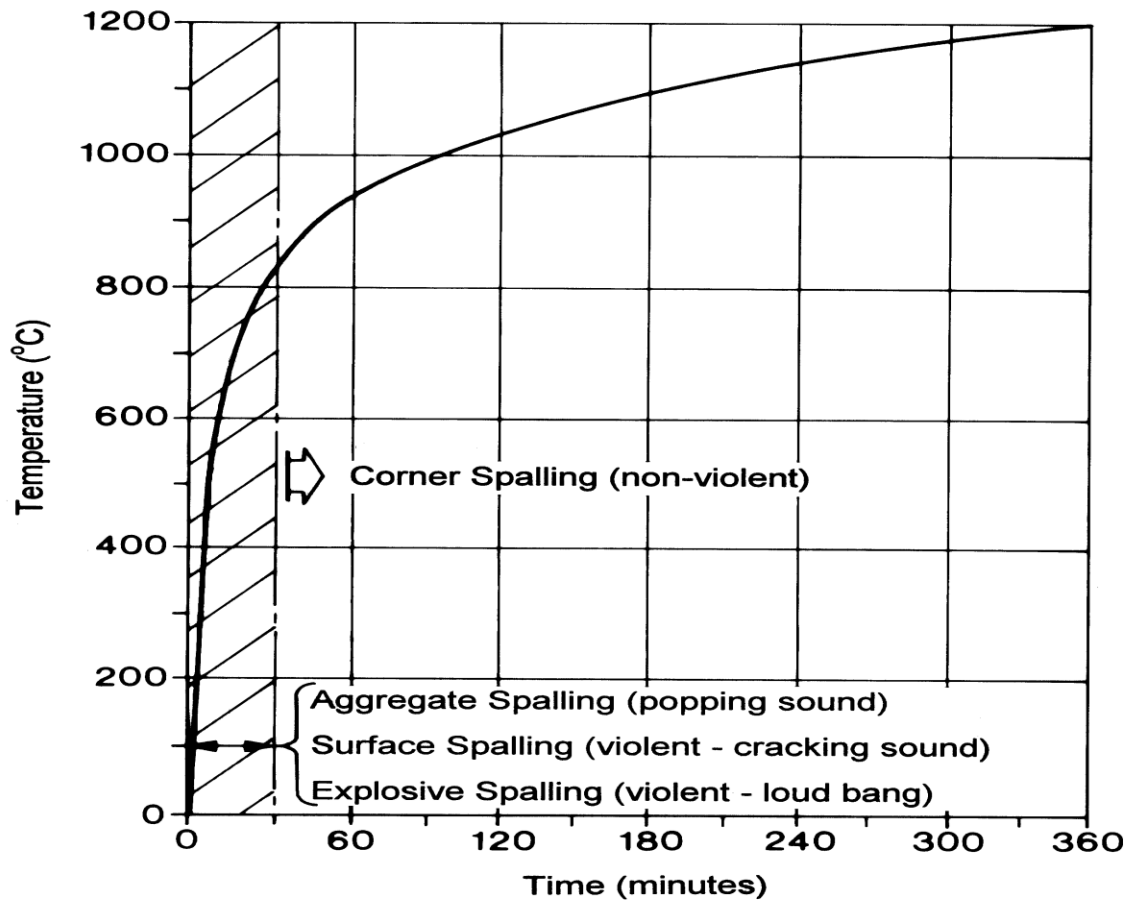


Figure 1.1 Shows time of occurrence of different types of spalling in a fire. (Source: Khoury and Anderberg, Fire Safety Design, (2000))

2.2.13: Sample of real fire Accident on concrete structures “Instructive fires”



Plate I. World Trade Center (source www.nceplus.co.uk)



Plate II. (Source: photos from J. C. López Agüí)



Plate III: Fire-retardant façades (source: CIMBETON)



Plate IV: The intense heat radiation of a fire forces the fire services to keep their distance from

2.3.0 LARGE-SCALE NON-STANDARD TESTS

The current review is focused on small-scale, non-standard structural fire resistance testing, with a particular emphasis that aimed at better understanding of response of concrete elements heated in electrical furnaces rather than real fires testing. However, in most cases standard fire tests have been used to perform structural fire resistance tests which clearly fall outside the scope of real and furnace testing procedures. Multiple tests on full-Scale fire tests were performed and are reported over a period of years in a single test structure. The brief overviews of the available tests on both steel and concrete structures are presented below. (British Steel, 1999).

2.3.1 Stuttgart-Vaihingen University Fire Test (1985)

One of the earliest experimental large-scale non-standard fire tests of the modern era was probably a test performed at the Stuttgart-Vaihingen University, Germany, in 1985. But only brief description of this test is available as reported on the Cardington tests (British Steel, 1999). This test was performed on the third floor of a four-story steel framed office building in which many different forms of steel and concrete composite elements had been used. The imposed fire was “natural” with a fire load consisting of wooden cribs.

2.3.2. Collins Street Test (circa 1994)

Shortly after the William Street tests, BHP performed a fire test in a steel-framed test building which was meant to simulate a section of a proposed multi-story building in Collins Street, also in Melbourne (British Steel, 1999).

2.3.3. Cardington Steel Building Tests (1996)

During 1996, an eight story composite steel-framed test building constructed at the Cardington test site of the UK Building Research Establishment. All beams were designed as simply supported, acting compositely with a 130 mm floor slab on steel decking. Beam-to-beam

connections were made using fin-plate connections and beam-to-column connections using flexible end plates. Sandbags were used to simulate gravity loads for typical office occupancy. A plan view of the test structure is given in Figure 1, which shows the locations and sizes of the various fire tests performed. Kathleen et al, (2012)

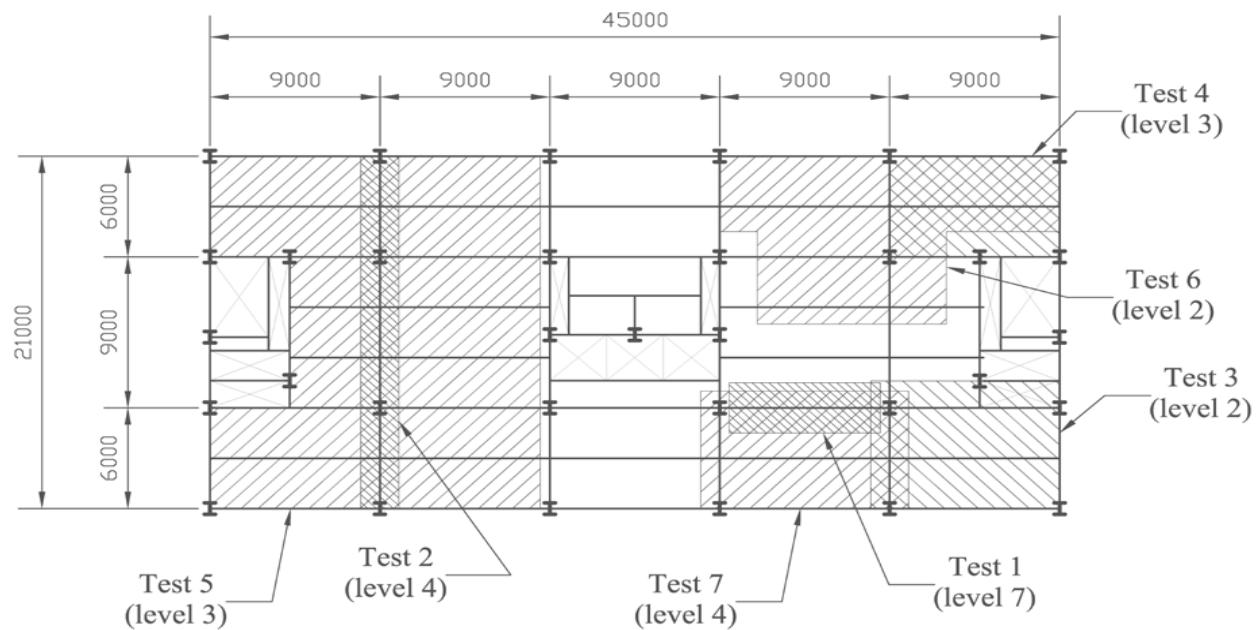


Fig. 2.2: Plan view of the Cardington Test Frame and locations of the seven fire tests performed in this structure (reproduced after Vassart and Zhao



Plate V



Plate VI

Plate V& VI: Fire tests at the Cardington steel building showing large deflections of the composite slab (Newman et al., 2000).

2.3.4. French Car Park Fire Tests (1998-2001)

A series of fire tests was conducted by the European Coal and Steel Council on an open-sided composite steel-framed car park structure. The objective of these tests was to show satisfactory structural fire performance without requiring fire protection to the structure. A single story structure 16 m × 32 m in plan with a height of 3 m was constructed from unprotected steel columns, steel-concrete composite beams, and a steel-concrete composite slab with a depth of 120 mm. The structure was loaded with real cars, and the fire load in the most severe of the three tests consisted of three real cars burning simultaneously and exposing a ‘significant area’ of the floor to the flames of the fire (Vassart and Zhao, 2011). This resulted in the steel beams above the fire being heated to more than 700°C.

Lu et al. (2011), from the same research group as the study above, have also presented details of a study on “the performance of an edge beam in a full-scale three-story steel framed building subjected to fire.”

2.3.5. Harbin Institute of Technology Tests (circa 2007 & 2010)

Dong and Prasad (2009a) present the details of a large-scale experimental study to “understand the performance of structural frames under fire loading” and to “to serve as a database with which to check and validate numerical models.” To do this, they constructed a two-dimensional, two-story, two-bay composite steel sway portal frame with fixed column base connections and subjected it to thermal loading by placing it inside a bespoke testing furnace while carrying sustained gravity loads. Full details of the testing facility are given by Dong and Prasad (2009a).

He results show that:

- (i) None of the columns in any of the three tests showed signs of local buckling or plastic hinge formation;

- (ii) Structural failure was observed in the beam-to-column connections, the composite beam and the concrete slab
- (iii) Local buckling of the steel beams near the end of their heated spans was observed, again this has been seen in many prior tests by others;
- (iv) Tensile cracking of the concrete slabs was observed near the span ends;

2.3.6. Mokrsko Fire Test (2008)

In September 2008, a large scale fire experiment was conducted in a purpose-built test structure by the Czech Technical University in Prague. The test was performed in Mokrsko, Czech Republic, and was designed to study the overall behavior of the test structure as well as its individual elements (Wald et al., 2010, Chlouba and Wald, 2009). Wald (2011) states that the main objective of this test was to observe the temperatures of partially encased header plate connections.

2.3.7. TU Munich Fire Tests (2010)

Staler et al. (2010) have briefly reported results of large-scale non-standard tests on a steel-concrete composite slab with the aim to understand the behavior of intermediate beams between two composite slab panels.

2.3.8. FRACOF Fire Test (circa 2008)

Vassart and Zhao (2011) present the results of two fire tests performed as part of the EU Commission-funded project Leonardo Da Vinci project on Fire Resistance Assessment of Partially Protected Composite Floors up to 120minutes (FRACOF). The building was one story high and all four columns and the perimeter beams were fire protected, whereas the internal secondary beams and composite slab were left unprotected. The results show that:

- (i) Failure of the integrity and insulation criteria were due to the formation of a crack across the composite slab due to premature failure of the reinforcing steel mesh in the composite slab;
- (ii) Proper overlapping of reinforcing steel mesh in the composite slab is essential to activate tensile membrane action and to ensure continuity of load transfer, particularly around columns;
- (iii) Concrete cracking at the edge of the floor was limited and had no influence on the integrity and insulation performance of the floor
- (iv) The floor behaved satisfactorily during the heating and cooling phases of the fire, as did all joints between steel members. (Vassart and Zhao 2011)

2.3.9. University of Ulster Fire Test (2010)

Vassart et al. (2010) present the results of a full-scale non-standard structural fire test performed by the University of Ulster in Northern Ireland in February 2010 with the aim to understand the development of tensile membrane action on solid beams. The test was ‘very successful’ in that the structure performed ‘as predicted.

2.3.10. COSSFIRE Full Scale Fire Test (circa 2008)

Vassart and Zhao (2011) conducted the second test called COSSFIRE test and was very similar to the FRACOF test. The structure was loaded using sandbags and subjected to 120 minutes of the ISO 834 (ISO, 1999). The key observations were that:

- (i) The deflection of the floor was more than 500 mm after 120 minutes,
- (ii) Local buckling was observed in unprotected secondary beam connections
- (iii) No failure of the edge connections between concrete slab and steel members was observed;

- (iv) Cracking of concrete around columns could have a negative impact on integrity criteria;
- (v) No significant cracking of the concrete slab was observed in the central part of the floor. (Vassart and Zhao 2011)

2.3.11. Veseli Fire Tests (2011)

Wald et al. (2011) present the results of two demonstration fire tests performed by the Czech Technical University in Prague in Veseli, Czech Republic during 2011 to investigate the ‘design of joints to composite columns for improved fire robustness.’ As of 2011, detailed experimental observations and conclusions from these tests are not available in the literature.

2.3.12. William Street Tests (circa 1992)

In the early 1990s a series of four fire tests were performed by BHP, Australia’s largest steel company, to investigate the structural fire performance of a specific steel-framed office building in Melbourne, Australia. It was desired to show that, with the installation of a light hazard sprinkler system and a non-fire rated suspended ceiling, fire protection could be removed from the structural steelwork of the beams and the steel deck floors (British Steel, 1999).

2.4 .0 Testing on Steel Connections in Fire

Reinforcing steel is much more sensitive to high temperatures than concrete. (API RP 579, 2000). In addition to the large-scale non-standard fire tests presented above, a number of testing programs have been undertaken in the past two decades to investigate the performance of connections in steel structures subject to both standard and non-standard heating scenarios. , Ding and Wang (2007) for information on fire testing of connections between steel beams and concrete-filled hollow structural sections, Chung et al. (2010) for tests on steel beam-to-column moment resisting connections, and Yuan et al. (2011)

2.4. 1. Fire Tests on Portal Frames and Sub-Frame Assemblies

Zhao and Shen (1999) experimentally studied the deformation behavior of unprotected two-dimensional, single-story, single span steel portal frames exposed to real fire conditions.. Loads were applied only to the column heads, with no applied loads on the top beams of the portal frames.

Wong et al. (1999) performed an experimental study, apparently the only of its kind, investigating the fire performance of steel portal frame buildings.

Santiago et al. (2008) present details of a unique testing facility and a series of tests on six steel ‘sub-frames’ under a ‘natural fire’ at the University of Coimbra, Portugal. The tests were intended to provide insight into the influence of various connection types on the behavior of steel sub-structures in fire.

2.5.0. ‘Non-Standard’ Testing in Furnaces

2.5.1. BRE Hollow-Core Slab Fire Test (2007)

In March 2007, two large-scale non-standard structural fire tests were performed by the UK Building Research Establishment at Middleborough, UK, to study the performance of hollow core concrete slab on steel beam flooring systems (Bailey and Lennon, 2008).

- (i) Van Herberghen and Van Damme (1983), who used a modified standard floor furnace to study the fire resistance of post-tensioned continuous flat floor slabs with unbonded tendons;
- (ii) Kordina (1997), who used a modified floor furnace to investigate the punching shear behavior of reinforced concrete flat slabs in fire conditions;
- (iii) Kelly and Purkiss (2008), who used an oversized floor furnace to study the fire resistance of simply supported long-span post-tensioned concrete slabs; and

- (iv) Li-Tang et al. (2008), who studied the structural fire behavior of model-scale three-span continuous unbounded post-tensioned concrete slab strips in a custom built furnace subject to the standard fire;
- (v) Zheng et al. (2010), who performed a series of tests on two-span continuous post-tensioned concrete slabs in a furnace with a central support built inside the heating chamber.
- (vi) Annerel et al. (2011), who used a modified standard floor furnace to perform punching shear tests on concrete slabs subject to the ISO 834 standard fire.

2.5.2. Hong Kong Fire Test (2010)

A large-scale non-standard fire test on unloaded concrete columns was performed in a real building in Hong Kong in August 2010 (Wong and Ng, 2011). The purpose of this test was to study the effects of water quenching on the fire performance of high strength concrete in a building fire. The results show that, in order to prevent spalling in high strength concrete columns, insulating materials should be used to restrict the heat transferred into the concrete

2.5.3. CCAA-CESARE Fire Test (2010)

Cement Concrete & Aggregates Australia (CCAA, 2010) reported on a single large-scale non-standard fire test performed simultaneously on both high strength concrete columns and post-tensioned concrete slabs. The purpose of the tests was to understand the fire performance of post-tensioned slabs and high strength concrete columns made from concrete using common Australian aggregates (with aggregate type being a known risk factor for explosive spalling fire).

From this test, the following conclusions were made:

- (i) The addition of fibers to the high strength mix had a dramatic effect in reducing the level of spalling

- (ii) The placement of column ties at closer spacing did not reduce the level of spalling;
- (iii) One of the post-tensioned slabs exhibited no spalling; whereas, an identical slab spalled at both ends with no obvious explanation.

2.5.4. TU Vienna Fire Tests (2011)

Ring et al. (2011) present the results of four large-scale non-standard fire tests on a ‘frame-like’ concrete structure in order to investigate the redistribution of loading within reinforced concrete structures subjected to fire (with a stated interest in buried infrastructure, i.e., tunnels). The tests were explicitly designed to provide data for the development, assessment, and validation of numerical tools for predicting the structural fire response of concrete structures, particularly for underground structures.

The results of these tests provide important data for validation of computational models and they again clearly demonstrate the benefits of incorporating polypropylene fibers to prevent explosive spalling of concrete in fire.

2.6.0 Modified Furnaces

A number of furnace testing facilities are also available globally, which can be used to study various aspects of the response of structures and structural components to fire in a variety of non-standard ways. The Fire Safety Engineering Research and Technology Centre at the University of Ulster, Northern Ireland (FireSERT), has a reconfigurable standard floor furnace, which has been modified to permit testing of specimens larger than would be typical. The furnace can also test beam-wall assemblies. (Wong and Ng, 2011).

The University Coimbra, Portugal, has recently developed a special ‘natural fire’ testing rig which is capable of testing steel sub-frame assemblies under exposure to gas burners with the

ability to spatially and temporally vary the thermal insult to loaded test assemblies (Santiago et al., 2008).

An example of a system, which is based on a pre-existing standard testing furnace, is the column testing facility at the Federal Institute for Materials Research and Testing (BAM), in Berlin, Germany. The BAM furnace is capable of producing an ISO 834 (ISO, 1999) fire on columns of up to 3.55 m in height. The furnace is fired by six oil burner and six electro-hydraulic control channels are used to actively control mechanical boundary conditions (two bending)

A similar approach has recently been taken at the National Research Council of Canada, Ottawa, Canada, where an existing standard column testing furnace has been upgraded to permit ‘hybrid’ testing of isolated structural elements by active control of applied loads during testing using coupled numerical analysis of the full structure in real time (Mostafaei, 2011). Heating is accomplished using electrical radiant heaters, and loading is accomplished using a complicated mechanism of restraining frames and loading jacks;

The furnace consists of 16 gas burners and is capable of simulating various standard fires, including ISO 834 (ISO, 1999) and hydrocarbon curves (CEN, 2002).

2.6.1 Wald (2011)

Wald, 2011) at a COST Action TU0904 Working Group meeting in April 2011 presented a paper that consists a summary, with source references and brief descriptions, of a number of large-scale non-standard furnace tests, all of which are described previously.

2.6.2 Vassart and Zhao (2011)

A publication was produced by Vassart and Zhao (2011) as a result of the EU Commission-funded project Leonardo Da Vinci project on Fire Resistance Assessment of Partially Protected Composite Floors (FRACOF). in France. It also presents evidence from the accidental real fires

at Broadgate and Churchill Plaza (discussed previously), and from small-scale fire resistance tests performed at the University of Manchester, UK (Bailey and Toh, 2006).

2.7.0. Structural Interactions and Asymmetry

According to Vodak *et al.*, (2004), the available test data from large-scale non-standard fire tests, while expensive, still cover only a small fraction of the possible structural configurations that are represented within the current global building stock let alone the highly optimized and sustainable buildings of the future. With a few notable exceptions, the majority of structural fire tests conducted to date, whether standard or non-standard, have studied regular, symmetric, highly idealized structures. Modern structures increasingly make use of irregular floor plates with varying span lengths, bay sizes, construction types, etc. The possible influence of irregular floor plans and complex building forms needs to be investigated, both experimentally and numerically, if performance-based structural fire engineering of both conventional and modern buildings is to be performed with confidence.

2.8.0. Failure Localizations

As already noted, when real structures fail in fires it is rarely for the reasons that might be expected on the basis of standard fire resistance testing. In many cases, failure is precipitated by some form of localized failure or distress, such as discrete cracking in concrete, rupture of tensile steel reinforcement, failure of a connection, local buckling of structural steelwork, shear (punching) failure of a concrete slab, etc. Unfortunately, the only way to observe and understand such failure localizations, which depend in virtually all cases on three dimensional interactions between elements of a structure during both heating *and* cooling, is to perform large-scale non-standard structural fire testing of real buildings. Only once the possible failure modes are known can they be rationally incorporated into computational models for full structure response. Felix *et al.*, (2010)

CHAPTER THREE

MATERIAL AND METHOD

3.1 Preamble.

This chapter essentially involves all the experiments conducted on the materials and the test specimens. The tests involved the preliminary and the major tests on the constituent materials (cement and aggregates), and the test specimens which involved the production, curing, heating, crushing and bending tests on the specimens. The laboratory procedures were carried out in accordance with the following standards: British Standard such as BS 12 (1999) BS 8110, (1989) American standard Test Material such as ASTM C187, 191 (1996), ASTM E 119(2008) and Eurocode 2-2, (2009) requirements. The average values of the tested parameters are compared to the standards and specification in order to come up with reasonable conclusions and recommendations on the effects of elevated temperature on concrete properties such as strength and stiffness. All the tests were carried out in the Department of Civil Engineering, Ahmadu Bello University, Zaria, with the exception of specimens heating which was carried out at Industrial Development Center, (IDC) Samaru, Zaria.

3.2 Tests on Cement

Portland cements are commonly characterized by their physical properties for quality control purposes. The challenge in physical property characterization is to develop physical tests that can satisfactorily characterize key parameters such as

- a. Fineness.
- b. Setting time tests
- c. Soundness test
- d. Consistency test
- e. Relative density (specific gravity) usually 3.15 for Portland cement.

The cement used in this study was the ordinary Portland cement (Dangwote cement) purchased at local shops in Samaru, Zaria. The above cement physical properties were tested in order to establish its suitability for the production of concrete specimens needed for this work.

3.2.1- Finenesses of Cement

The fineness test was carried out in accordance with the British Standard (BS 12) 1999. Table 3.1 shows the test results of fineness of Dangwote ordinary Portland cement used in this study.

Table 3.1. Fineness of the Portland cement

S/N	Fineness	Average value of fineness(g)
1	0.04	0.036
2	0.04	
3	0.03	

3.2.2- Standard Consistency Test

The consistency test on the Portland cement was carried out in accordance with EN 196-3 and 6 (1994 and 1995) BS 12:1996 and BS 8110, (1997) provisions. Table 3.2 shows the result. The raw data is as shown in Appendix A-21

Table 3.2 show the consistency result of ordinary Portland cement

Test No	Consistency
1	7mm
2	6mm

Raw data at Appendix A-21

3.2.3- Setting Time Tests

The initial and final setting time test was carried out in accordance with British Standard BS12 and A.S.T.M: C-187, and A.S.T.M : C-191-70 .The results of the initial and final setting times are the average of the six different tests and are presented in Table 3.3 below. The raw data is as shown in Appendix A-22

Table 3.3: shows the average of initial and final setting time of OPC.

Initial setting Time (min)	Final Setting Time (min)
104	270

The raw data is as shown in Appendix A-22

3.2.4- Soundness Test

The test was conducted using ‘Le Chatelier’ method of measuring expansion of cement in accordance with EN 196-3: 1994 and BS EN, 196-2, (1995). Table3.4 shows the result of the soundness test. The raw data is as shown in Appendix A-23

Table3.4. Soundness of cement paste

S/No	Average expansion(mm)
A	2.3
B	2.0
C	2.5
Overall average	2.26

The raw data is as shown in Appendix A-23

3.3 Tests on Aggregates

3.3.1- Aggregates Particle size Distribution)

(a) Fine Aggregates:

The fine aggregate was the naturally occurring clean sand obtained from River Rimi in Katsina State which was then supplied to the Department of Civil Engineering, Ahmadu Bello University Zaria for experimental purposes. The Sieve analyses on fine aggregates were conducted in accordance with BS 812 (1975). The result is shown on table 3.5 and the raw data is shown in

Appendix A-24-29

Table 3.5: Sieve analysis of fine aggregates

Sieve size mm	% passing cum.
10.00	100
5,00	98.3
2.36	94.4
1.18	85.4
6 μ	74.2
3 μ	45.4
1.5 μ	14.7
0.75 μ	4.9

(b) Coarse Aggregates

The grading on individual size (10mm, 14mm, 20mm) of coarse aggregates was carried out separately and then combined together and graded as well. The coarse aggregates were sourced from the Mother Cat quarry located at Rimi local government in Katsina State as shown on plate I. below and then supplied to the Department of Civil Engineering, Ahmadu Bello University Zaria for experimental activities. The analysis was conducted in accordance with BS 812 – 1(1975). The result is as shown on table 3.6-3.18 below. Plate I shows the aggregates sourcing



Plate VII: Sourcing of Coarse aggregates (20mm, 14mm and 10mm) at Mother

Table 3.6: Sieve analysis of 10mm coarse aggregate

Sieve sizes (mm)	Cum % passing
14.00	100
10.00	92.2
5.00	21.9
12.36	2.0

The raw data is as shown in Appendix A-25

Table 3.7: Sieve analysis of 14mm coarse aggregates

Sieve size mm	Cum.% passing
20.00	100
14.00	90.7
10.00	46.4
5.00	24.0

The raw data is as shown in Appendix A-26

Table 3.8: Sieve analysis of 20mm coarse aggregates

Sieve size mm	Cum. % passing
37.50	100
20.00	92.3
14.00	49.0
10.00	18.2
5.00	3.2

The raw data is as shown in Appendix A- 29

3.3.2. Specific Gravity of Aggregates

The test was carried out in accordance with BS 1377:-3 1990, BS 812 (1985) and I.S 2386-1.(1999). The raw data is in **Appendix A-30 -34**. The result is as presented in table 3.9 below.

Table 3.9: Specific gravity of the coarse and the fine aggregates

S/N	Aggregate sizes(mm)	Average G_s g/m ³	Average Value G_s g/m ³
1	20	2.660	2.655
2	14	2.672	
3	10	2.663	
4	Stone dust	2.665	
5	River sand	2.615	

3.3.3. Aggregate Crushing Value

This test was carried out in accordance with BS 882 and BS 812. The combined aggregates are used in experimental work to determine the aggregates Crushing Value. The raw data is presented in **Appendix A-35**. Table3.10 shows the result of aggregates crushing value.

Table3. 10 Result of aggregate crushing value for coarse aggregates.

Test No	Sample A (gm)	Sample B (gm)	Averages (%)	Overall Average (%)
1	2419.5	428.7	17.7%	Result = 18.7%
2	2787.3	581.2	20.9%	
3	2754.0	482.3	17.5%	

3.3 4. Water Absorption of Aggregates

The test was performed in accordance with BS-882 (1985). Just like aggregates crushing value, Water absorption is also carried out on the combined aggregates and the result is as shown in table 3. 11 **(See appendix A-36-39 for detail)**

Table 3.11, Result of the water absorption test of the combined aggregate

Size of aggregates (mm)	Average value of water absorption%
20	1.5
14	1.4
10	1.2
Sharp sand	0.6
Average value=1.43	

3.4 Mixing Water

The water used throughout casting operation of the specimens was municipal water that is drinkable and portable and has no definite taste or odour. The water used in carrying out the preliminary test was sourced from main Laboratory of the Department of Civil Engineering, Ahmadu Bello University, Zaria.

3.5. Test on Fresh Concrete

3.5.1- Workability

The slump was carried out in accordance with ASTM C143 (1999) using graduated still ruler. Some of the result is shown in table 3. 12 below

Table 3.12: Shows the slumps of various grades of concrete.

Test no	Slumps (cm)	Averages	Average
1	6.2, 6.9, 6.5	6.5cm	5.5cm
2	6.4, 6.7, 6.8	6.6cm	
3	5.7, 4.9, 4.7	5.1cm	
4	5.0, 4.9, 5.9	5.2cm	
5	4.5, 4.7, 4.5	4.7cm	
6	4.9, 5.3, 4.9	5.0cm	

3.6 Grade 30 Concrete Trial Mix Design proportion

This method follows the general requirements for both mass and reinforced concrete EN 197-1 and ESS 1658/1988. The nominal size of aggregates used is 20mm.

The ratio of the constituent materials used is **1: 1 – : 3**. With water cement ratio of 0.45.

With the above mix ratio and water cement ratio using absolute volume, the strength of different concrete obtained using different aggregates size and that of combined aggregates is as shown in table 3.13 below as trial mix (M_1 , M_2 , M_3 and M_4) where M_1 stands for combined aggregate concrete, M_2 for 10mm aggregates, M_3 for 14mm aggregates concrete and M_4 stands for 20mm aggregates concrete

Table3.13 Compressive strength of concrete trial mix

Age (days)	M ₁ (N/mm ²)	M ₂ N/mm ²	M ₃ N/mm ²	M ₄ N/mm ²	W/C Ratio %
7days	20.50	20.1	20.0	19.1	0.45
14Days	27.52	27.0	26.9	26.5	
28Days	30.71	30.1	30.0	30.0	

The result there is an average of three sample cubes for each concrete type.

3.7 Preparation / Production of Concrete Test Specimens

Using the trial mix ratio, the specimen for the tests were produced. The test specimens were divided in to two groups. Group 1 consist of total number of 1066 standard 150mm cubes including control specimens and they were used to evaluate the residual compressive strength of concrete under different elevated temperatures and exposure time. As for the group II, a total number of 81 square concrete prisms of (150 x150 x750mm) were produced using 10mm deformed steel bars with yield strength of 460N/mm² as reinforcement with a view of evaluating the stiffness as well as the bending strength of normal (combined aggregates) reinforced concrete under different elevated temperature, exposure time and different concrete cover. In the production process, the following steps were followed:

1. Both coarse and fine aggregates were measured out in designed proportion and thoroughly mixed on hard and clean concrete surface.

2. Cement was also measured out and mix throughly for 5- 10 minutes with the aggregates before adding water.
3. Slump tests were performed on each of the concrete mixes according to ASTM C143
4. After mixing, the fresh concrete was then filled in to iron moulds and compacted in 2-layers, using 16mm rod tapping it 25 times.
5. The cast specimens were covered with a plastic sheet to keep moisture from evaporating
6. The specimens were then allowed to cure in the moulds for 24hrs before demoulding
7. The demoulded concrete specimens were placed and kept in water for the specified curing times until testing age. Plates II show the preparation activities



Plate VIII: Preparation of test Specimens (Prisms)

3.8 Curing of Specimens.

The specimens were demolded, immersed and kept in to curing tank at room temperature until testing age after twenty four hours of casting. In this work, cubes specimens were cured for one, three, seven, fourteen and twentyeight days respectively (1,3,7,14,&28 days). Whereas, the prism specimens were cured for the period of twentyeight days only. The curing operation is shown below.



Plate IX: Curing of Concrete Specimens

3.9 Heating (Cube Specimens)

Heating of specimens was carried out in accordance with Australia /New Zealand Standard (2005) and BS1881-116 (1983). As earlier said, One thousand and sixty six 150mm standard cubes (1066) specimen sample were produced, sixty six (66) of them are the control specimens as earlier said and were subjected to different temperatures in electrical furnace, the remaining 1006 were subjected to different temperatures in electrical furnace. Heating of specimens was carried at Industrial Development Center (I.D.C.) Samaru Zaria. The cubes were subjected to the following varying temperatures range from: 100°C , 300°C , 500°C , 700°C and 900°C in an increment of 200°C . For each of the temperature above, three cubes were subjected to that temperature for the different duration: 30mins, 60mins, 90mins and 120mins with increment of 30minutes.

That means for each targeted temperature, twelve (12) cubes were heated. This implies that, three (3) cubes for each duration. After heating the concrete specimens for the targeted duration, the cubes specimens are removed and allowed to cool for twenty four hours (24 hrs) before

crushing test is conducted. Plate x shows loading of specimens in to the heating equipment



Plate x: Loading of specimens in to heating chamber (electric furnace)

3.10 Heating (Square prism specimens)

The square prism specimens were produced for only twenty eight days age. Eighty specimens were produced and subjected to different temperature of 100°C , 300°C and 600°C , at two different duration time of one hour (1hr) and two hours (2hrs). Three different thicknesses of concrete covers to reinforcement were employed in order to confirm the validity or otherwise of fire resistance provision by codes as major design parameter of concrete in fire. Thus, a square prism of 28 days were produced with concrete covers of 20mm, 25mm and 30mm and tested for three temperatures of 100°C , 300°C and 600°C at different duration of one hour (1hr) and two hours (2hrs). Thus, thirty six (36) square prisms were used for the flexural strength only and another thirty six (36) square prisms were produced for bending strength determination. Thus, making a total number of seventy two (72) square prisms. The remaining eight (8) specimens were kept in case of replacement.

The summary of heating duration and concrete cover at different temperature are shown in table 3.14 to 3.15. Twelve number prisms were used for each temperature. Number of concrete prisms is as shown on tables 3.14 and 3.16 below.

Table 3.14 : Summary of heating duration and concrete cover at 100⁰c

Duration(mins)	Concrete Cover (mm)
1hr	20
2hrs	
1hr	25
2hrs	
1hr	30
2hrs	

Table 3.15: Summary of heating duration and concrete cover at 300⁰c

Duration(mins)	Concrete Cover (mm)
1hr	20
2hrs	
1hr	25
2hrs	
1hr	30
2hrs	

Table 3.16 Summary of heating duration and concrete cover at 600⁰c

Duration(mins)	Concrete Cover (mm)
1hr	20
2hrs	
1hr	25
2hrs	
1hr	30
2hrs	

3.11 Specimens Weighing

After all the specimens were remolded, cured and attained the required age for testing, their respective weight were taken both before heating and after heating except the control specimens which were only weigh once. The Weighing of the specimens were carried out at the department of civil engineering laboratory at Ahmadu Bello University, Zaria using weighing machine. The difference in weight of the specimens was recorded and the result as presented on tables 3.17 to 3. 116.

3.11. (i): Thirty Minutes Exposure Time (30minutes)

Table 3.17 to 3. 41 represent result of the Percentage weight reduction of concrete 1-day, 3-days, 7-days, 14-days and 28-days old concrete at 100⁰c, 300⁰c, 500⁰c, 700⁰c and 900⁰c at 30minuets exposure time

Table 3.17: Percentage weight reduction of 1-day old at 100⁰c/ 30minuets exposure

1-Day @100⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.7	8.6	0.1	1.2
M_2	8.6	8.5	0.1	1.25
M_3	8.5	8.3	0.2	1.6
M_4	8.5	8.3	0.2	2.3

Table 3.18: Percentage weight reduction of 3-days old at 100⁰c/ 30minuets exposure

3-Days @100⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.8	8.72	0.08	0.97
M_2	8.7	8.1	0.1	1.1
M_3	8.7	8.55	0.15	1.0
M_4	8.5	8.35	0.15	1.3

Table 3.19: Percentage weight reduction of 7-days old at 100⁰c/ 30minuets exposure

7-Days old @ 100⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.8	8.2	0.6	5.8
M_2	8.5	8.0	0.7	8.0
M_3	8.4	7.8	0.8	9.0
M_4	8.4	7.9	0.8	11.2

Table 3.20: Percentage weight reduction of 14-days old at 100⁰c/ 30minuets exposure

14-Days @ 100⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.5	7.9	0.7	7.0
M_2	8.7	8.0	0.8	9.2
M_3	8.5	7.7	0.8	9.7
M_4	8.5	7.6	0.9	11.0

Table 3.21: Percentage weight reduction of 28-Days old at 100⁰c/ 30minuets exposure

28-Days @ 100⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.8	8.1	0.7	0.8
M_2	8.5	7.7	0.8	0.9
M_3	8.7	7.8	0.9	0.10
M_4	8.7	7.8	0.9	0.11

Table 3.22: Percentage weight reduction of 1- Day old at 300⁰c/ 30minuets exposure

1- Days 300⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.8	8.2	0.6	6.8
M_2	8.7	8.57	0.13	1.45
M_3	8.7	7.55	0.15	1.70
M_4	8.7	7.55	0.15	1.70

Table 3.23: Percentage weight reduction of 3- Days old at 300^{0c}/ 30minuets exposure

3- Days 300^{0c}				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.8	8.69	0.11	1.25
M_2	8.8	8.68	0.12	1.40
M_3	8.7	7.55	0.15	1.70
M_4	8.7	7.55	0.15	1.70

Table 3.24: Percentage weight reduction of 7- Days old at 300^{0c}/ 30minuets exposure

7- Days @300^{0c}				
<i>Types of concrete</i>	Weight Before heating(kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.8	8.7	0.1	1.14
M_2	8.7	8.58	0.12	1.40
M_3	8.7	7.55	0.15	1.70
M_4	8.7	7.55	0.15	1.70

Table 3.25: Percentage weight reduction of 14- Days old at 300⁰c/ 30minuets exposure

14- Days @300⁰c				
<i>Types of concrete</i>	WT Before heating	WT after heating kg	Difference kg	% reduction in Wt
M_1	8.8	8.70	0.10	1.10
M_2	8.8	8.69	0.11	1.25
M_3	8.8	8.68	0.12	1.40
M_4	8.7	8.58	0.12	1.40

Table 3.26: Percentage weight reduction of 28- Days old at 300⁰c/ 30minuets exposure

28- Days @300⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.9	8.81	0.09	1.00
M_2	8.8	8.70	0.10	1.10
M_3	8.7	8.59	0.11	1.25
M_4	8.8	8.68	0.12	1.25

Table 3.27: Percentage weight reduction of 1- Day old 500^{0c}/ 30minuets exposure

1- Day @500^{0c}				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.9	8.85	0.15	1.70
M_2	8.8	8.6	0.20	2.2
M_3	8.8	8.6	0.20	2.2
M_4	8.7	8.48	0.22	2.5

Table 3.28 :Percentage weight reduction of 3- Days old 500^{0c}/ 30minuets exposure

3- Days @500^{0c}				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Diffirence (kg)	% reduction in Weight
M_1	8.9	8.86	0.14	1.60
M_2	8.8	8.76	0.14	1.60
M_3	8.8	8.75	0.15	1.70
M_4	8.7	8.65	0.15	1.70

Table 3.29: Percentage weight reduction of 7- Days old at 500⁰c/ 30minuets exposure

7- Days @500⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.9	8.78	0.12	1.40
M_2	8.8	8.76	0.14	1.60
M_3	8.8	8.76	0.14	1.60
M_4	8.7	8.56	0.14	1.60

Table 3.30: Percentage weight reduction of 14- Days old at 500⁰c/ 30minuets exposure

14- Days @500⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.8	8.69	0.11	1.30
M_2	8.8	8.68	0.12	1.40
M_3	8.9	8.78	0.12	1.40
M_4	8.8	8.76	0.14	1.50

Table 3.31: Percentage weight reduction of 28- Days at 500^{0c}/ 30minuets exposure

28- Days @500⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.9	8.79	0.11	1.25
M_2	8.8	8.69	0.11	1.30
M_3	8.7	8.59	0.11	1.30
M_4	8.7	8.59	0.11	1.30

Table 3.32: Percentage weight reduction of 1- Day old at 700^{0c}/ 30minuets exposure

1- Day @700⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	9.1	8.88	0.22	2.5
M_2	8.9	8.65	0.25	2.8
M_3	8.8	8.55	0.25	2.8
M_4	8.8	8.52	0.28	3.1

Table 3.33: Percentage weight reduction of 3- Days old at 700^{0c}/ 30minuets exposure

3- Days @700^{0c}				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.9	8.69	0.21	2.35
M_2	8.9	8.78	0.22	2.5
M_3	8.9	8.77	0.23	2.60
M_4	8.8	8.56	0.24	2.72

Table 3.34: Percentage weight reduction of 7- Days old at 700^{0c}/ 30minuets exposure

7- Days @700^{0c}				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction Weight
M_1	8.8	8.6	0.20	2.2
M_2	8.8	8.59	0.21	2.30
M_3	8.7	8.49	0.21	2.30
M_4	8.7	8.58	0.22	2.5

Table 3.35: Percentage weight reduction of 14- Days old at 700^{0c}/ 30minuets exposure

14- Days @700⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	W Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.8	8.72	0.18	2.01
M_2	8.8	8.6	0.20	2.2
M_3	8.7	8.5	0.20	2.2
M_4	8.7	8.49	0.21	2.30

Table 3.36: Percentage weight reduction of 28- Days old at 700^{0c}/ 30minuets exposure

28- Days @700⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.9	8.89	0.17	1.91
M_2	8.8	8.61	0.19	2.1
M_3	8.8	8.61	0.19	2.1
M_4	8.7	8.5	0.20	2.2

Table 3.37: Percentage weight reduction of 1- Day old at 900^{0c}/ 30minuets exposure

1- Day @900^{0c}				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	9.0	8.64	0.26	2.8
M_2	8.9	8.63	0.28	3.10
M_3	8.8	8.52	0.28	3.10
M_4	8.7	8.41	0.29	3.30

Table 3.38: Percentage weight reduction of 3- Days old at 900^{0c}/ 30minuets exposure

3- Days @900^{0c}				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.9	8.69	0.21	2.30
M_2	8.7	8.47	0.23	2.60
M_3	8.7	8.47	0.23	2.60
M_4	9.0	8.64	0.26	2.8

Table 3.39: Percentage weight reduction of 7- Day old at 900^{0c}/ 30minuets exposure

7- Days @900^{0c}				
<i>Types of concrete</i>	Weight Before heating (kg)	W Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.8	8.60	0.20	2.10
M_2	8.7	8.5	0.20	2.2
M_3	8.8	8.59	0.21	2.35
M_4	8.7	8.48	0.22	2.4

Table 3.40: Percentage weight reduction of 14- Days old at 900^{0c}/ 30minuets exposure

14- Days @900^{0c}				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.8	8.60	0.18	2.00
M_2	8.5	8.3	0.2	2.10
M_3	8.7	8.50	0.2	2.10
M_4	8.7	8.5	0.21	2.3

Table 3.41: Percentage weight reduction of 28- Days old at 900^{0c}/ 30minuets exposure

28- Days @900^{0c}				
<i>Types of concrete</i>	Weight Before heating (kg)	W Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.5	8.3	0.2	2.10
M_2	8.8	8.59	0.21	2.35
M_3	8.7	8.48	0.22	2.4
M_4	9.0	8.66	0.24	2.6

3.11. (ii). Sixty Minutes Exposure Time (60minutes)

Table 3.41 to 3. 66 represent result of the Percentage weight reduction of concrete 1-day, 3-days, 7-days, 14-days and 28-days old concrete at 100^{0c}, 300^{0c}, 500^{0c}, 700^{0c} and 900^{0c} at 30minuets exposure time

Table 3.42: Percentage weight reduction of 1-day old at 100⁰c/ 60minuets exposure

1-Day @100⁰c @				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.9	8.75	0.15	1.68
M_2	8.8	8.6	0.2	2.30
M_3	8.8	8.6	0.2	2.30
M_4	8.7	8.45	0.25	2.90

Table 3.43: Percentage weight reduction of 3-days old at 100⁰c/ 60minuets exposure

3-Days @100⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	9.0	8.87	0.13	1.4
M_2	8.8	8.65	0.15	1.79
M_3	8.7	8.50	0.2	2.2
M_4	8.7	8.50	0.2	2.2

Table 3.44: Percentage weight reduction of 7-days old at 100⁰c/ 60minuets exposure

7-Days old @ 100⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.8	8.7	0.1	1.2
M_2	8.8	8.68	0.12	1.40
M_3	8.7	8.58	0.12	1.40
M_4	8.7	8.57	0.13	1.50

Table 3.45: Percentage weight reduction of 14-days old at 100⁰c/ 60minuets exposure

14-Days @ 100⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.8	8.71	0.09	1.10
M_2	8.8	8.7	0.1	1.2
M_3	8.8	8.68	0.12	1.40
M_4	8.8	8.68	0.12	1.40

Table 3.46: Percentage weight reduction of 28-Days old at 100⁰c/ 60minuets exposure

28-Days @ 100⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.9	8.81	0.09	1.0
M_2	8.8	8.71	0.09	1.0
M_3	8.7	8.61	0.07	1.0
M_4	8.8	8.7	0.1	1.2

Table 3.47: Percentage weight reduction of 1- Day old at 300⁰c/ 60minuets exposure

1- Days 300⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.7	8.45	0.25	2.90
M_2	8.8	8.52	0.28	3.1
M_3	8.8	8.52	0.28	3.1
M_4	8.7	8.40	0.3	3.4

Table 3.48: Percentage weight reduction of 3- Days old at 300^{0c}/ 60minuets exposure

3- Days 300^{0c}				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.8	8.68	0.22	2.5
M_2	8.9	8.77	0.23	2.60
M_3	8.8	8.67	0.23	2.60
M_4	8.8	8.56	0.24	2.72

Table 3.49: Percentage weight reduction of 7- Days old at 300^{0c}/ 60minuets exposure

7- Days @300^{0c}				
<i>Types of concrete</i>	Weight Before heating(kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.7	8.49	0.21	2.30
M_2	8.8	8.68	0.22	2.5
M_3	8.8	8.68	0.22	2.5
M_4	8.8	8.67	0.23	2.60

Table 3.50: Percentage weight reduction of 14- Days old at 300⁰c/ 60minuets exposure

14- Days @300⁰c				
<i>Types of concrete</i>	WT Before heating	WT after heating kg	Difference kg	% reduction in Wt
M_1	8.8	8.61	0.19	2.1
M_2	8.7	8.5	0.20	2.2
M_3	8.7	8.5	0.20	2.2
M_4	8.7	8.5	0.20	2.2

Table 3.51: Percentage weight reduction of 28- Days old at 300⁰c/ 60minuets exposure

28- Days @300⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.9	8.81	0.09	1.00
M_2	8.8	8.70	0.10	1.10
M_3	8.7	8.59	0.11	1.25
M_4	8.8	8.68	0.12	1.25

Table 3.52: Percentage weight reduction of 1- Day old 500^{0c}/ 60minuets exposure

1- Day @500^{0c}				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.8	8.55	0.25	2.9
M_2	8.8	8.50	0.30	3.4
M_3	8.7	8.40	0.30	3.4
M_4	8.7	8.38	0.32	3.7

Table 3.53 Percentage weight reduction of 3- Days old 500^{0c}/ 60minuets exposure

3- Days @ 500^{0c}				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	9.0	8.77	0.13	2.60
M_2	8.9	8.65	0.25	2.9
M_3	8.8	8.55	0.25	2.9
M_4	8.8	8.55	0.25	2.9

Table 3.54 Percentage weight reduction of 7- Days old at 500⁰c/ 60minuets exposure

7- Days @500⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.8	8.6	0.20	2.2
M_2	8.7	8.5	0.20	2.2
M_3	8.7	8.58	0.22	2.5
M_4	8.7	8.56	0.24	2.7

Table 3.55: Percentage weight reduction of 14- Days old at 500⁰c/ 60minuets exposure

14- Days @500⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	W Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.8	8.72	0.18	2.0
M_2	8.8	8.72	0.18	2.0
M_3	8.8	8.6	0.20	2.2
M_4	8.7	8.5	0.20	2.2

Table 3.56: Percentage weight reduction of 28- Days old at 500⁰c/ 60minuets exposure

28- Days @500⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Diffidence (kg)	% reduction in Weight
M_1	8.9	8.75	0.15	1.7
M_2	8.8	8.62	0.18	2.0
M_3	8.8	8.62	0.18	2.0
M_4	8.7	8.52	0.18	2.0

Table 3.57 Percentage weight reduction of 1- Day old concrete at 700⁰c/ 60minuets exposure

1- Day @700⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.8	8.52	0.28	3.1
M_2	8.7	8.41	0.29	3.30
M_3	8.8	8.51	0.29	3.30
M_4	8.7	8.4	0.3	3.4

Table 3.58 Percentage weight reduction of 3- Days old at 700⁰c/ 60minuets exposure

3- Days @700⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.9	8.65	0.25	2.9
M_2	8.8	8.54	0.26	3.0
M_3	8.8	8.54	0.26	3.0
M_4	8.7	8.44	0.26	3.0

Table 3.59: Percentage weight reduction of 7- Days old at 700⁰c/ 60minuets exposure

7- Days @700⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction Weight
M_1	8.9	8.78	0.22	2.5
M_2	8.8	8.67	0.23	2.7
M_3	8.7	8.66	0.24	2.8
M_4	8.7	8.56	0.24	2.8

Table 3.60 Percentage weight reduction of 14- Days old at 700⁰c/ 60minuets exposure

14- Days @700⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	W Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.9	8.69	0.21	2.30
M_2	8.7	8.49	0.21	2.30
M_3	8.8	8.68	0.22	2.5
M_4	8.7	8.47	0.23	2.6

Table 3.61 Percentage weight reduction of 28- Days old at 700⁰c/ 60minuets exposure

28- Days @700⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.8	8.61	0.19	2.1
M_2	8.8	8.61	0.19	2.1
M_3	8.7	8.5	0.20	2.2
M_4	8.7	8.5	0.20	2.2

Table 3.62 Percentage weight reduction of 1- Day old concrete at 900⁰c/ 60minuets exposure

1- Day @900⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.9	8.6	0.3	3.4
M_2	8.9	8.58	0.32	3.6
M_3	8.8	8.48	0.32	3.6
M_4	8.8	8.45	0.35	3.9

Table 3.63 Percentage weight reduction of 3- Days old at 900⁰c/ 60minuets exposure

3- Days @900⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.9	8.63	0.27	3.31
M_2	8.9	8.6	0.3	3.4
M_3	8.7	8.4	0.3	3.4
M_4	8.7	8.38	0.32	3.6

Table 3.64 Percentage weight reduction of 7- Day old concrete at 900⁰c/ 60minuets exposure

7- Days @900⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	W Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.8	8.55	0.25	2.9
M_2	8.8	8.54	0.26	3.0
M_3	8.7	8.44	0.26	3.0
M_4	8.8	8.52	0.28	3.1

Table 3.65 Percentage weight reduction of 14- Days old at 900⁰c/ 60minuets exposure

14- Days @900⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	9.0	8.66	0.24	2.6
M_2	8.8	8.46	0.24	2.6
M_3	8.8	8.45	0.25	2.8
M_4	8.7	8.44	0.26	3.0

Table 3.66 Percentage weight reduction of 28- Days old at 900^{0c}/ 60minuets exposure

28- Days @900^{0c}				
<i>Types of concrete</i>	Weight Before heating (kg)	W Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.7	8.48	0.22	2.4
M_2	8.8	8.68	0.22	2.5
M_3	8.7	8.47	0.23	2.6
M_4	8.8	8.45	0.25	2.8

3.11.(iii). Ninety Minutes Exposure Time (90minutes)

Table 3.66 to 3. 91 represent result of the Percentage weight reduction of concrete 1-day, 3-days, 7-days, 14-days and 28-days old concrete at 100^{0c}, 300^{0c}, 500^{0c}, 700^{0c} and 900^{0c} at 30minuets exposure time

Table 3.67 Percentage weight reduction of 1-day old at 100^oc/ 90minuets exposure

1-Day @100^oc @				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.9	8.65	0.25	2.90
M_2	8.8	8.52	0.28	3.1
M_3	8.8	8.52	0.28	3.1
M_4	8.7	8.40	0.3	3.4

Table 3.68 Percentage weight reduction of 3-days old at 100^oc/ 90minuets exposure

3-Days @100^oc				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.8	8.67	0.23	2.60
M_2	8.8	8.56	0.24	2.72
M_3	8.7	8.44	0.26	2.9
M_4	8.7	8.44	0.26	2.9

Table 3.69 Percentage weight reduction of 7-days old at 100⁰c/ 90minuets exposure

7-Days old @ 100⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.9	8.79	0.21	2.4
M_2	8.8	8.57	0.23	2.6
M_3	8.8	8.57	0.23	2.6
M_4	8.8	8.57	0.23	2.6

Table 3.70 Percentage weight reduction of 14-days old at 100⁰c/ 90minuets exposure

14-Days @ 100⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.9	8.7	0.2	2.2
M_2	8.8	8.6	0.2	2.2
M_3	8.7	8.59	0.21	2.4
M_4	8.8	8.59	0.21	2.4

Table 3.71 Percentage weight reduction of 28-Days old at 100⁰c/ 90minuets exposure

28-Days @ 100⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.9	8.72	0.18	2.0
M_2	8.8	8.61	0.19	2.1
M_3	8.8	8.61	0.19	2.1
M_4	8.7	8.5	0.2	2.2

Table 3.72 Percentage weight reduction of 1- Day old at 300⁰c/ 90minuets exposure

1- Days 300⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.9	8.56	0.34	3.9
M_2	8.8	8.44	0.36	4.1
M_3	8.8	8.43	0.37	4.2
M_4	8.7	8.32	0.38	4.4

Table 3.73 Percentage weight reduction of 3- Days old at 300⁰c/ 90minuets exposure

3- Days 300⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.8	8.50	0.3	3.4
M_2	8.8	8.48	0.32	3.60
M_3	8.8	8.46	0.34	3.9
M_4	8.7	8.36	0.34	3.9

Table 3.74 Percentage weight reduction of 7- Days old at 300⁰c/ 90minuets exposure

7- Days @300⁰c				
<i>Types of concrete</i>	Weight Before heating(kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.8	8.52	0.28	3.1
M_2	8.8	8.51	0.29	3.3
M_3	8.8	8.51	0.29	3.3
M_4	8.8	8.50	0.3	3.4

Table 3.75 Percentage weight reduction of 14- Days old at 300⁰c/ 90minuets exposure

14- Days @300⁰c				
<i>Types of concrete</i>	WT Before heating	WT after heating kg	Difference kg	% reduction in Wt
M_1	9.0	8.74	0.26	2.9
M_2	8.8	8.53	0.27	3.0
M_3	8.8	8.53	0.27	3.0
M_4	8.7	8.42	0.28	3.1

Table 3.76 Percentage weight reduction of 28- Days old at 300⁰c/ 90minuets exposure

28- Days @300⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	9.0	8.77	0.13	2.60
M_2	8.8	8.66	0.24	2.7
M_3	8.7	8.56	0.24	2.7
M_4	8.7	8.56	0.24	2.7

Table 3.77 Percentage weight reduction of 1- Day old 500⁰c/ 90minuets exposure

1- Day @500⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.8	8.43	0.37	4.2
M_2	8.8	8.43	0.37	4.2
M_3	8.7	8.30	0.4	4.6
M_4	8.7	8.30	0.4	4.6

Table 3.78 Percentage weight reduction of 3- Days old 500⁰c/ 90minuets exposure

3- Days @500⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.8	8.44	0.36	4.1
M_2	8.8	8.44	0.36	4.1
M_3	8.8	8.43	0.37	4.2
M_4	8.8	8.43	0.37	4.2

Table 3.79 Percentage weight reduction of 7- Days old at 500⁰c/ 90minuets exposure

7- Days @500⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.8	8.45	0.35	3.9
M_2	8.8	8.44	0.36	4.0
M_3	8.8	8.44	0.36	4.0
M_4	8.7	8.33	0.37	4.2

Table 3.80 Percentage weight reduction of 14- Days old at 500⁰c/ 90minuets exposure

14- Days @500⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	W Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.8	8.48	0.32	3.60
M_2	8.8	8.48	0.32	3.60
M_3	8.8	8.47	0.33	3.8
M_4	8.8	8.47	0.33	3.8

Table 3.81 Percentage weight reduction of 28- Days old at 500⁰c/ 90minuets exposure

28- Days @500⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Diffidence (kg)	% reduction in Weight
M_1	8.8	8.5	0.3	3.3
M_2	8.7	8.4	0.3	3.3
M_3	8.8	8.48	0.32	3.60
M_4	8.8	8.48	0.32	3.60

Table 3.82 Percentage weight reduction of 1- Day old concrete at 700⁰c/ 90minuets exposure

1- Day @700⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.9	8.45	0.45	5.2
M_2	8.8	8.3	0.5	5.7
M_3	8.8	8.3	0.5	5.7
M_4	8.7	8.26	0.54	6.4

Table 3.83 Percentage weight reduction of 3- Days old at 700⁰c/ 90minuets exposure

3- Days @700⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.8	8.38	0.42	4.8
M_2	8.8	8.37	0.43	4.9
M_3	8.9	8.45	0.45	5.2
M_4	8.9	8.45	0.45	5.2

Table 3.84 Percentage weight reduction of 7- Days old at 700⁰c/ 90minuets exposure

7- Days @700⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction Weight
M_1	8.9	8.59	0.41	4.6
M_2	8.8	8.38	0.42	4.8
M_3	8.8	8.38	0.42	4.8
M_4	8.7	8.27	0.43	4.9

Table 3.85 Percentage weight reduction of 14- Days old at 700⁰c/ 90minuets exposure

14- Days @700⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	W Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.9	8.61	0.39	4.3
M_2	8.8	8.4	0.4	4.6
M_3	8.8	8.4	0.4	4.6
M_4	8.8	8.38	0.42	4.8

Table 3.86 Percentage weight reduction of 28- Days old at 700⁰c/ 90minuets exposure

28- Days @700⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.8	8.36	0.34	3.9
M_2	8.8	8.34	0.36	4.1
M_3	8.7	8.32	0.38	4.4
M_4	8.7	8.32	0.38	4.4

Table 3.87 Percentage weight reduction of 1- Day old at 900⁰c/ 90minuets exposure

1- Day @900 ⁰ c 2-				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.9	8.1	0.8	8.9
M_2	8.9	8.05	0.85	9.6
M_3	8.9	8.0	0.9	10.1
M_4	8.7	7.8	0.9	10.1

Table 3.88 Percentage weight reduction of 3- Days old at 900⁰c/ 90minuets exposure

3- Days @900 ⁰ c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.8	8.05	0.75	8.5
M_2	8.8	8.0	0.8	8.6
M_3	8.8	8.0	0.8	8.9
M_4	8.8	7.93	0.87	9.8

Table 3.89 Percentage weight reduction of 7- Day old concrete at 900⁰c/ 90minuets exposure

7- Days @900⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	W Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	9.0	8.26	0.74	8.2
M_2	8.8	8.14	0.76	8.6
M_3	8.8	8.14	0.76	8.6
M_4	8.8	7.9	0.8	9.1

Table 3.90 Percentage weight reduction of 14- Days old at 900⁰c/ 90minuets exposure

14- Days @900⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.9	8.2	0.7	7.9
M_2	8.8	8.08	0.72	8.2
M_3	8.8	8.14	0.76	8.6
M_4	8.8	8.14	0.76	8.6

Table 3.91 Percentage weight reduction of 28- Days old concrete at 900^{0c}/ 90minuets exposure

28- Days @900⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	W Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.8	8.17	0.63	7.2
M_2	8.8	8.15	0.65	7.5
M_3	8.7	8.0	0.7	8.1
M_4	8.8	8.08	0.72	8.2

3.11.(iv): One Hundred and Twenty Minutes Exposure Time (120minutes)

Table 3.91 to 3. 116 represent result of the Percentage weight reduction of concrete 1-day, 3-days, 7-days, 14-days and 28-days old concrete at 100^{0c}, 300^{0c}, 500^{0c}, 700^{0c} and 900^{0c} at 30minuets exposure time

Table 3.92 Percentage weight reduction of 1-day old at 100°C / 120minuets exposure

1-Day @100°C @				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.8	8.5	0.5	5.7
M_2	8.6	8.0	0.6	5.8
M_3	8.5	7.9	0.6	7.0
M_4	8.5	7.9	0.6	7.0

Table 3.93 Percentage weight reduction of 3-days old at 100°C / 120minuets exposure

3-Days @100°C				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.8	8.3	0.5	6.0
M_2	8.7	8.1	0.6	7.0
M_3	8.7	8.0	0.7	8.0
M_4	8.5	7.5	0.8	10

Table 3.94 Percentage weight reduction of 7-days old at 100⁰C/ 120minuets exposure

7-Days old @ 100⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.6	8.0	0.6	5.0
M_2	8.5	8.0	0.7	8.0
M_3	8.4	7.8	0.8	9.0
M_4	8.4	7.9	0.8	11.2

Table 3.95 Percentage weight reduction of 14-days old at 100⁰C / 120minuets exposure

14-Days @ 100⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.5	7.9	0.7	7.0
M_2	8.5	7.8	0.7	8.2
M_3	8.5	7.7	0.8	9.4
M_4	8.4	7.6	0.8	9.6

Table 3.96 Percentage weight reduction of 28-Days old at 100⁰C / 120minuets exposure

28-Days @ 100⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.6	8.3	0.7	3.4
M_2	8.5	8.1	0.8	4.7
M_3	8.5	8.0	0.9	5.8
M_4	8.5	7.9	0.9	7.0

Table 3.97 Percentage weight reduction of 1- Day old at 300⁰C / 120minuets exposure

2- Days 300⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.8	8.2	0.6	6.8
M_2	8.7	8.0	0.7	8.1
M_3	8.5	7.8	0.7	8.1
M_4	8.5	7.6	0.9	10.5

Table 3.98 Percentage weight reduction of 3- Days old at 300⁰C / 120minuets exposure

3- Days 300⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.5	7.7	0.8	6.1
M_2	8.6	8.0	0.5	6.7
M_3	8.5	7.5	1.0	8.1
M_4	8.5	7.5	1.0	9.3

Table 3.99 Percentage weight reduction of 7- Days old at 300⁰C / 120minuets exposure

7- Days @300⁰c				
<i>Types of concrete</i>	Weight Before heating(kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.9	8.0	0.7	5.7
M_2	8.8	8.0	0.8	6.5
M_3	8.8	8.0	0.85	7.2
M_4	8.6	7.57.5	0.95	8.3

Table 3.100 Percentage weight reduction of 14- Days old at 300⁰C / 120minuets exposure

14- Days @300⁰c				
<i>Types of concrete</i>	WT Before heating	WT after heating kg	Difference kg	% reduction in Wt
M_1	8.9	8.0	0.7	5.1
M_2	8.8	8.0	0.8	6.0
M_3	8.8	7.1	0.9	6.5
M_4	8.6	7.7	1.1	8.0

Table 3.101 Percentage weight reduction of 28- Days old at 300⁰C / 120minuets exposure

28- Days @300⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.8	8.0	0.7	4.5
M_2	8.8	8.0	0.8	5.0
M_3	8.6	8.0	0.8	6.3
M_4	8.6	7.6	1.0	7.3

Table 3.102 Percentage weight reduction of 1- Day old 500°C / 120 minutes exposure

1- Day @500°C				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.9	8.0	0.73	7.9
M_2	8.8	8.0	0.75	9.1
M_3	8.8	8.0	0.8	9.1
M_4	8.6	7.5	0.8	9.4

Table 3.103 Percentage weight reduction of 3- Days old 500°C / 120 minutes exposure

3- Days @500°C				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.9	8.0	0.7	7.0
M_2	8.8	7.9	0.8	8.5
M_3	8.8	7.9	0.9	10.9
M_4	8.6	7.6	1.0	11.6

Table 3.104 Percentage weight reduction of 7-Days old at 500°C / 120minuets exposure

7- Days @500⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.9	8.0	0.7	7.9
M_2	8.8	8.0	0.8	9.1
M_3	8.8	8.0	0.9	10.1
M_4	8.6	7.57.5	1.0	11.8

Table 3.105 Percentage weight reduction of 14-Days old at 500°C/120minuets exposure

14- Days @500⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	W Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.9	8.0	0.8	7.9
M_2	8.8	8.0	0.9	9.1
M_3	8.7	7.8	0.9	10.3
M_4	8.6	7.7	1.0	10.5

Table 3.106 Percentage weight reduction of 28-Days old at 500⁰C/120minuets exposure

28- Days @500⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.9	8.0	0.9	10.1
M_2	8.8	7.8	1.0	11.4
M_3	8.8	7.7	1.1	12.1
M_4	8.6	7.5	1.1	12.8

Table 3.107 Percentage weight reduction of 1- Day old at 700⁰C / 120minuets exposure

1- Day @700⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.9	7.8	0.7	12.3
M_2	8.8	7.6	0.7	13.6
M_3	8.6	7.4	0.8	13.9
M_4	8.5	7.5	0.9	12.8

Table 3.108 Percentage weight reduction of 3- Days old at 700⁰C /120minuets exposure

3- Days @700⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.9	7.9	0.7	11.2
M_2	8.8	7.7	0.75	12.5
M_3	8.7	7.6	0.8	12.5
M_4	8.6	7.5	0.8	12.8

Table 3.109 Percentage weight reduction of 7- Days old concrete at700⁰C / 120minuets exposure

7- Days @700⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction Weight
M_1	8.9	8.0	0.7	7.9
M_2	8.8	8.0	0.8	9.1
M_3	8.8	8.0	0.8	9.1
M_4	8.6	7.5	1.1	12.8

Table 3.110 Percentage weight reduction of 14-Days old at 700⁰C/120minuets exposure

14- Days @700⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	9.2	8.1	0.9	10.1
M_2	8.9	7.8	0.9	10.2
M_3	8.9	8.0	1.0	11.1
M_4	8.6	7.5	1.1	12.7

Table 3.111 Percentage weight reduction of 28-Days old at 700⁰C /120minuets exposure

28- Day @700⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	9.1	8.0	0.8	9.1
M_2	8.8	8.0	0.8	9.1
M_3	8.8	8.0	0.8	9.1
M_4	8.6	7.5	1.1	12.8

Table 3.112 Percentage weight reduction of 1-Days old at 900⁰C /120minuets exposure

1- Day @900⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	W Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	9.1	8.0	0.8	9.1
M_2	8.8	8.0	0.8	9.1
M_3	8.8	8.0	0.8	9.1
M_4	8.6	7.5	1.1	12.8

Table 3.113 Percentage weight reduction of 3- Days old at 900⁰C /120minuets exposure

3- Days @900⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.9	7.9	1.0	11.2
M_2	8.8	7.7	1.1	12.5
M_3	8.8	8.0	1.1	11.9
M_4	8.6	7.4	1.2	13.9

Table 3.114: Percentage weight reduction of 7- Day old at 900⁰C / 120minuets exposure

7- Days @900⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.9	7.8	1.1	12.3
M_2	8.7	7.6	1.1	12.6
M_3	8.5	7.2	1.3	15.1
M_4	8.4	7.1	1.3	15.5

Table 3.115 Percentage weight reduction of 14-Days old at 900⁰C/120minuets exposure

14- Days @900⁰c				
<i>Types of concrete</i>	Weight Before heating (kg)	Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.9	8.1	0.8	9.0
M_2	8.5	7.2	1.3	15.1
M_3	8.8	8.0	0.8	9.1
M_4	8.4	7.1	1.3	15.5

Table 3.116 Percentage weight reduction of 28-Days old at 900⁰C/120minuets exposure
28- Days @900⁰c

<i>Types of concrete</i>	Weight Before heating (kg)	W Weight after heating (kg)	Difference (kg)	% reduction in Weight
M_1	8.6	7.4	0.8	13.1
M_2	8.5	7.2	1.3	15.1
M_3	8.5	7.2	1.4	15.1
M_4	8.5	7.2	1.3	15.1

3.12 Tests On Hardened Specimens

3.12.1. Concrete Compression Test

The tests specimens after weighing were crushed in accordance with ASTM C297 in the departments of Civil Engineering Ahmadu Bello University, Zaria. Denison Universal testing machine was used in crushing the test specimens as shown below.



Plate xi: Compression Test of cubes specimens

To determine the strength of concrete that had undergone high temperature, the prepared cubes specimens were subjected to different temperatures and durations, and tested for Compressive strength in accordance to BS 1881, part 116, 1983. The cubes specimens were tested for ages of 1, 3, 7, 14 and 28 days of age.

3.12.1(a). Residual Compressive Strength of Concrete

To see the effect of temperatures on concretes strength, the cubes specimens are subjected to compressive load after being heated in electrical furnace to different elevated temperature. An average of three cubes is used for each temperature and exposure period. The results are presented in tables. 3.117 to 3.136 below

Table 3.117 Residual compressive strength of 1-day age Concrete cubes heated for 30minutes exposure

Mix type Temperature⁰c	M ₁ N/mm ²	M ₂ N/mm ²	M ₃ N/mm ²	M ₄ N/mm ²
Room temp.	5.7	5.5	5.4	5.1
100	6.8	6.1	5.7	5.2
300	4,3	3.9	3.6	3.0
500	1.4	1.0	0.5	0.4
700	0.3	0.1	0.1	0.1
900	0.0	0.0	0.0	0.0

Table 3.118 Residual compressive strength of 3--day's age concrete cubes heated for 30minutes

Mix Type Temperature⁰c	M ₁ N/mm ²	M ₂ N/mm ²	M ₃ N/mm ²	M ₄ N/mm ²
Room Temperature	16.2	16.0	15.9	15.6
100	19.6	18.2	17.5	16.1
300	13.3	612.3	11.1	10.5
500	7.9	6.7	6.0	5.5
700	1.6	1.1	0.8	0.6
900	00.0	0.0	0.0	0.0

Table 3.119 Residual compressive strength of 7--day's age concrete cubes heated for 30minutes

Mix Type Temperature⁰c	M ₁ N/mm ²	M ₂ N/mm ²	M ₃ N/mm ²	M ₄ N/mm ²
Room Temp.	19.9	19.2	19.0	18.6
100	23.3	20.5	19.4	18.6
300	19.3	18.9	18.7	14.9
500	10.6	9.6	9.1	8.6
700	4.0	3.3	2.5	2.0
900	0.8	0.6	0.1	0.1

Table 3.120 Residual compressive strength of 14--days age concrete cubes heated for 30minutes

Mix Type Temperature⁰c	M ₁ N/mm ²	M ₂ N/mm ²	M ₃ N/mm ²	M ₄ N/mm ²
Room Temp.	24.0	23.4	23.0	22.6
100	26.8	24.6	23.7	22.6
300	23.8	20.7	20.0	18.3
500	13.2	12.2	11.5	10.9
700	8.6	7.7	7.7	6.8
900	2.4	2.3	1.4	0.7

Table.3.121 Residual compressive strength of 28--days age concrete cubes heated for 30minutes

Mix Type Temperature °C	M ₁ N/mm ²	M ₂ N/mm ²	M ₃ N/mm ²	M ₄ N/mm ²
Room Temp.	31.7	31.0	30.6	30.1
100	32.3	31.3	30.6	30.1
300	31.7	30.4	27.2	25.6
500	18.7	18.3	17.4	15.4
700	12.1	11.8	11.3	9.6
900	5.4	5.0	2.5	3.3

Table 3.122 Residual compressive strength of 1--day age concrete cubes heated for 60minutes

Mix Type Temperature °C	M ₁ N/mm ²	M ₂ N/mm ²	M ₃ N/mm ²	M ₄ N/mm ²
Room Temp.	5.9	5.6	5.6	5.2
100	6.8	5.6	5.4	4.7
300	3.8	3.4	3.0	2.5
500	1.0	0.1	0.5	0.4
700	0.0	0.0	0.0	0.0
900	0.0	0.0	0.0	0.0

Table 3.123 Residual compressive strength of 3-days age concrete cubes heated for 60minutes

Mix Type Temperature °C	M ₁ N/mm ²	M ₂ N/mm ²	M ₃ N/mm ²	M ₄ N/mm ²
Room Temp	16.0	16.0	15.8	15.4
100	17.6	16.8	15.8	15.1
300	14.7	14.2	13.8	12.6
500	6.9	6.4	5.7	5.1
700	2.6	2.2	2.1	1.7
900	0.5	0.3	0.2	0.2

Table 3.124 Residual compressive strength of 7--day's age concrete cubes heated for 60minutes

Mix Type Temperature °C	M ₁ N/mm ²	M ₂ N/mm ²	M ₃ N/mm ²	M ₄ N/mm ²
Room Temp	21.0	20.0	20.1	19.6
100	22.5	20.4	20.1	19.6
300	20.1	18.4	18.0	16.9
500	10.7	9.6	9.0	8.4
700	6.7	6.0	5.0	4.5
900	2.3	1.6	1.2	0.8

Table 3.125 Residual compressive strength of 14--days age concrete cubes heated for 60minutes

Mix Type Temperature⁰c	M ₁ N/mm ²	M ₂ N/mm ²	M ₃ N/mm ²	M ₄ N/mm ²
Room Temp.	23.6	23.0	22.6	22.3
100	25.0	23.0	22.6	22.3
300	21.7	20.0	18.8	17.8
500	11.1	10.4	9.5	8.9
700	10.0	8.3	7.2	6.7
900	2.4	1.8	1.4	1.4

Table 3.126 Residual compressive strength of 28-day's age concrete cubes heated for 60 minutes

Mix Type Temperature⁰c	M ₁ N/mm ²	M ₂ N/mm ²	M ₃ N/mm ²	M ₄ N/mm ²
Room Temp.	31.0	30.4	30.0	30.0
100	33.8	31.0	30.0	30.4
300	29.5	27.1	25.5	24.9
500	15.5	14.6	14.1	12.9
700	12.7	12.0	12.0	9.6
900	3.7	3.0	2.4	2.1

Table 3.127 Residual compressive strength of 1--day age concrete cubes heated for 90minutes

Mix Type Temperature⁰c	M ₁ N/mm ²	M ₂ N/mm ²	M ₃ N/mm ²	M ₄ N/mm ²
Room Temp.	6.1	5.8	5.2	5.0
100	6.1	5.0	4.9	4.6
300	3.6	3.5	2.8	2.4
500	0.9	0.5	0.4	0.4
700	0.0	0.0	0.0	0.0
900	0.0	0.0	0.0	0.0

Table 3.128 Residual compressive strength of 3--days age concrete cubes heated for 90minutes

Mix Type Temperature⁰c	M ₁ N/mm ²	M ₂ N/mm ²	M ₃ N/mm ²	M ₄ N/mm ²
Room Temp.	16.1	15.7	15.1	15.0
100	17.1	15.7	15.1	15.0
300	11.8	10.9	9.7	8.6
500	6.4	5.2	4.1	3.6
700	1.6	0.6	0.2	0.0
900	0.0	0.0	0.0	0.0

Table 3.129 Residual compressive strength of 7--days age concrete cubes heated for 90minutes

Mix Type Temperature⁰c	M ₁ N/mm ²	M ₂ N/mm ²	M ₃ N/mm ²	M ₄ N/mm ²
Room Temp.	20.0	19.4	19.1	19.0
100	20.8	19.8	18.3	17.1
300	17.0	15.5	7.6	6.8
500	10.0	9.1	7.8	6.8
700	4.4	3.9	3.4	3.0
900	0.2	0.0	0.0	0.0

Table 3.130 Residual compressive strength of 14--days age concrete cubes heated for 90minutes

Mix Type Temperature⁰c	M ₁ N/mm ²	M ₂ N/mm ²	M ₃ N/mm ²	M ₄ N/mm ²
Room Temp.	24.6	23.4	23.0	22.7
100	25.6	23.4	21.6	19.7
300	21.4	19.6	18.9	17.5
500	13.0	11.7	10.8	9.8
700	5.9	4.9	4.4	4.3
900	1.7	0.9	0.9	0.5

Table 3.131 Residual compressive strength of 28--day's age concrete cubes heated for 90 minutes

Mix Type Temperature⁰c	M ₁ N/mm ²	M ₂ N/mm ²	M ₃ N/mm ²	M ₄ N/mm ²
Room Temp.	31.2	31.0	30.5	30.0
100	31.2	31.0	29.0	27.0
300	28.4	27.3	24.8	23.0
500	16.5	15.5	14.3	12.5
700	7.5	6.5	5.8	4.8
900	3.4	2.2	2.1	1.2

Table 3.132: Residual compressive strength of 1--day age concrete cubes heated for 120minutes

Mix Type Temperature⁰c	M ₁ N/mm ²	M ₂ N/mm ²	M ₃ N/mm ²	M ₄ N/mm ²
Room Temp.	6.0	5.7	5.7	5.4
100	5.7	4.9	4.6	4.2
300	3.6	3.4	3.1	2.5
500	0.6	0.6	0.6	0.4
700	0.1	0.0	0.0	0.0
900	0.0	0.0	0.0	0.0

Table 3.133: Residual compressive strength of 3--days age concrete cubes heated for 120minutes

Mix Type Temperature⁰c	M ₁ N/mm ²	M ₂ N/mm ²	M ₃ N/mm ²	M ₄ N/mm ²
Room Temp	15.4	15.0	15.0	15.0
100	13.8	13.0	12.1	11.4
300	10.8	10.5	9.0	8.4
500	4.2	3.2	3.2	1.8
700	1.1	1.0	0.6	0.2
900	0.0	0.0	0.0	0.0

Table 3.134: Residual compressive strength of 7--day's age concrete cubes heated for 120minutes

Mix Type Temperature⁰c	M ₁ N/mm ²	M ₂ N/mm ²	M ₃ N/mm ²	M ₄ N/mm ²
Room Temp.	20.0	19.2	19.0	19.0
100	18.0	16.7	15.2	14.3
300	14.0	12.7	11.6	10.8
500	7.8	6.5	5.6	5.3
700	2.4	1.5	0.8	0.2
900	0.6	0.4	0.4	0.2

Table 3.135: Residual compressive strength of 14--day's age concrete cubes heated for 120minutes

Mix type Temperature⁰c	M ₁ N/mm ²	M ₂ N/mm ²	M ₃ N/mm ²	M ₄ N/mm ²
Room Temp.	24.5	24.0	23.1	23.0
100	22.0	20.1	18.4	17.3
300	18.4	16.8	16.1	15.4
500	10.8	9.4	8.3	7.4
700	4.9	4.3	3.4	3.2
900	0.7	0.5	0.4	0.2

Table 3.136: Residual compressive strength of 28--day's age concrete cubes heated for 120minutes

Mix Type Temperature⁰c	M ₁ N/mm ²	M ₂ N/mm ²	M ₃ N/mm ²	M ₄ N/mm ²
Room Temp.	31.0	30.8	30.8	30.4
100	27.9	25.6	24.6	24.3
300	24.8	23.4	22.0	19.8
500	14.0	14.0	11.4	9.1
700	8.1	5.5	5.0	4.3
900	2.2	1.2	1.2	0.6

3.12.2. Three- Points Bending Test

The three point bending test is another test on hardened concrete. The test was carried out in accordance to Australia /New Zealand Standard (2005), TM-009, 2005 and ASTM C469 (1998).

The loading process is as shown below.



Plate xii: Three Point Bending Test in progress with crack propagation on specimen's surface

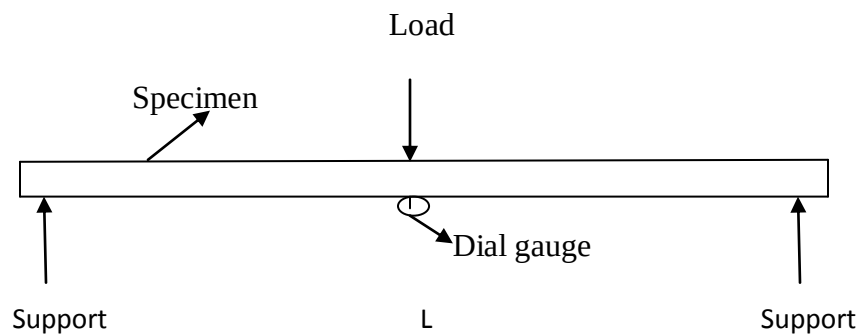


Fig.3.1 Three Point Bending Test Loading Arrangements

Using the hydraulic jack attached with dial gauge, and is pump until it reaches a reading of one graduation on the hydraulic jack meter equivalent to 2716.2 N/mm^2 and the corresponding dial

gauge reading at the bottom is read off. This continues until the specimen fail. The crack propagation of one of the specimen is as shown in the plate above. The whole test on prisms was carried out in the Department of Civil Engineering Ahmadu Bello University, Zaria.

3.12.2(a) Residual Deflection

Tables 3.137 to 3.146 present the residual deflections for reinforced concrete prisms with the corresponding loading for three different concrete covers (20mm, 25mm and 30mm) at **100⁰c**, **300⁰c** and **600⁰c** exposure target elevated temperatures at 60minutes and 120minutes exposure periods.

Table 3.137 Residual deflection of different concrete cover @ 100⁰c, 60mins duration

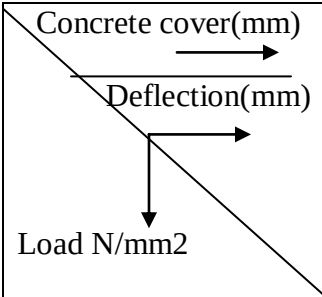
	20mm	25mm	30mm
	residual deflection (mm)	Residual deflection (mm)	Residual deflection (mm)
2716.8	6.6	5.6	5.1
10865.1	11.2	9.5	6.8
16296.6	16.9	12.3	11.2
21729.1	23.0	20.0	14.6

Table 3.138 Residual deflection of different concrete cover @ 300⁰c, 60mins duration

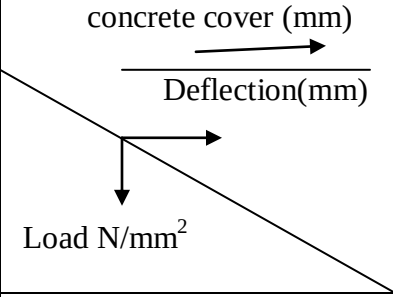
 concrete cover (mm) Deflection(mm) Load N/mm ²	20mm	25mm	30mm
	residual deflection (mm)	residual deflection (mm)	residual deflection (mm)
2716.2	9.5	8.2	7.2
10864.8	20	13.6	9.6
16297.2	26.2	19.5	14.2
21729.6	30.0	26.0	20

Table 3.139 Residual deflection of different concrete cover @ 600⁰c, 60mins duration

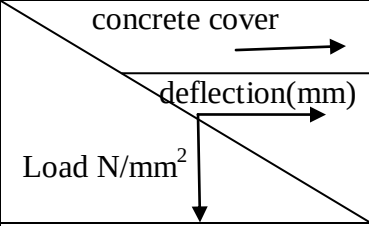
 concrete cover deflection(mm) Load N/mm ²	20mm	25mm	30mm
	residual def.mm	residual def.mm	residual def.mm
2716.2	14.2	12.9	10.4
10864.8	21.3	20.0	18.9
16297.2	29.0	26	22
21729.6	31.0	27.0	24

Table 3.140 Residual deflection of different concrete cover @ 100⁰c, 120mins duration

concrete cover → Deflection (mm) → ↓ Loading(N/mm²)	20mm	25mm	30mm
	residual deflection (mm)	residual deflection (mm)	residual deflection (mm)
2716.2	11.0	9.0	6.5
10864.8	13.6	12.0	9.1
16297.2	20.7	18.0	1.8
21729.6	29.6	22.0	20

Table 3.141 Residual deflection of different concrete cover @ 300⁰c, 120mins duration

Concrete cover (mm) → Deflection(mm) → ↓ Load N/mm2	20mm	25mm	30mm
	residual deflection (mm)	residual deflection (mm)	residual deflection (mm)
2716.2	15.0	12.0	9.0
10864.8	22.0	19.0	16.0
16297.2	26.0	23.0	18.4
21729.6	35.0	32.0	28.0

Table 3.142 Residual deflection of different concrete cover @ 600⁰c, 120mins duration

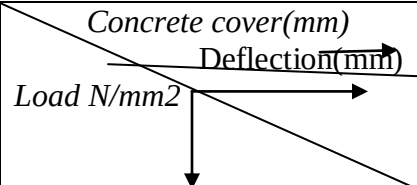
	20mm	25mm	30mm
	residual def.mm	residual def.mm	residual def.mm
2716.2	17.8	16.2	12.0
10864.8	24.0	22.0	20.0
16297.2	29.0	26.0	23.0
21729.6	40.0	34.0	31.0

Table3.143 Variation of elastic modulus with cover at 60mins exposure time

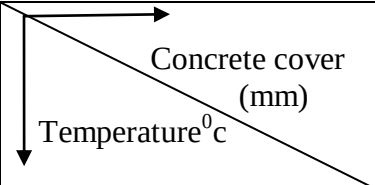
Loading N/mm ² ↓		Modulus (E)N/mm ² ▲			Exposure period 60mins
		20	25	30	
2716.2	Ambient temp (Control)	2436	2780	2920	
10864.8	100	2268	2420	2568	
16297.2	300	1932	2031	2420	
21729.6	600	1011	1293	1432	

Table 3.144 Variation of residual elastic modulus with cover at 120mins exposure time.

Loading N/mm^2	Concrete Cov(mm) Temperature $^{\circ}\text{C}$	Modulus (E) N/mm^2			120mins Exposure period
		30	25	20	
2716.2	Ambient temp (Control)	2911	2776	2440	
10864.8	100	2211	2100	2002	
16297.2	300	2000	1700	1520	
21729.6	600	1234	1100	1020	

Table 3.145 variation of bending strength with cover at 60mins exposure time

Loading N/mm^2	Concrete Cover (mm) Temperature $^{\circ}\text{C}$	Bending strength N/mm^2			60mins Exposure period
		20	25	30	
2716.2	Ambient temp (control)	7.16	7.38	7.42	
10864.8	100	5.2	5.8	6.2	
16297.2	300	4.4	4.9	5.2	
21729.6	600	2.6	3.0	3.4	

Table 3.146 Variation of bending strength with cover at 120mins

Loading N/mm ² ↓	Cover size (mm) → Temp. °C →	Bending strength N/mm ² ↓			120mins Exposure period
		20	25	30	
2716.2	Ambient temp (control)	7.1	7.31	7.4	
10864.8	100	6.2	6.6	7.1	
16297.2	300	5.0	5.4	6.1	
21729.6	600	3.1	3.5	4.3	

CHAPTER FOUR

RESULTS AND DISCUSSIONS

4.1 Preamble

The test result from both preliminary test on materials as well as that of the major test on heated concrete specimens is hereby discussed so as to arrive at reasonable conclusion and recommendation. The average values of all the tested parameters are compared to the standards and specification requirements.

4.2 Cement Tests

The test results of fineness, consistency, setting time and Soundness carried on the ordinary Portland cement were established and the results are presented in table 3.1, 3.2, 3.3 and 3.4 respectively in chapter three. The average values of the parameters tested mentioned above are compared to the standards and specification as shown in table 4.1 below.

Table 4.1 Comparism of Cement Parameters with Standard Specifications

S/N	Cement Parameter	Value (unit)	Standard Values DIN 1164-6 and BS 8110-1,1997
1	Fineness	361mm ² / (g)	600mm ² / (g)
2	Consistency	30	33%max
3	Initial Setting Time	104(minutes)	45min (min)
4	Final Setting Time	255(minutes)	375min(max)
5	Soundness	2.30(mm)	10mm (max)

From table 4.1 shown above, the cement parameters tested satisfied the Codes of practice. Thus one can say that the cement used in this research is of good quality. The cement is of good quality and complies with the standard specification.

4.3 Aggregates Test

As earlier explained, four types of aggregates are used. This include

- (a) Fine aggregates,
- (b) Ten (10mm) nominal Size coarse aggregates
- (c) Fourteen (14mm) nominal Size coarse aggregates
- (d) Twenty (20mm) nominal Size coarse aggregates

4.3.1 Particle Size Distribution

(a) Fine aggregates

The sieve analysis (particle size distribution) of fine aggregates used in this research is presented in table 3.5 of chapter three and the graph is as presented in figure 4.1 below

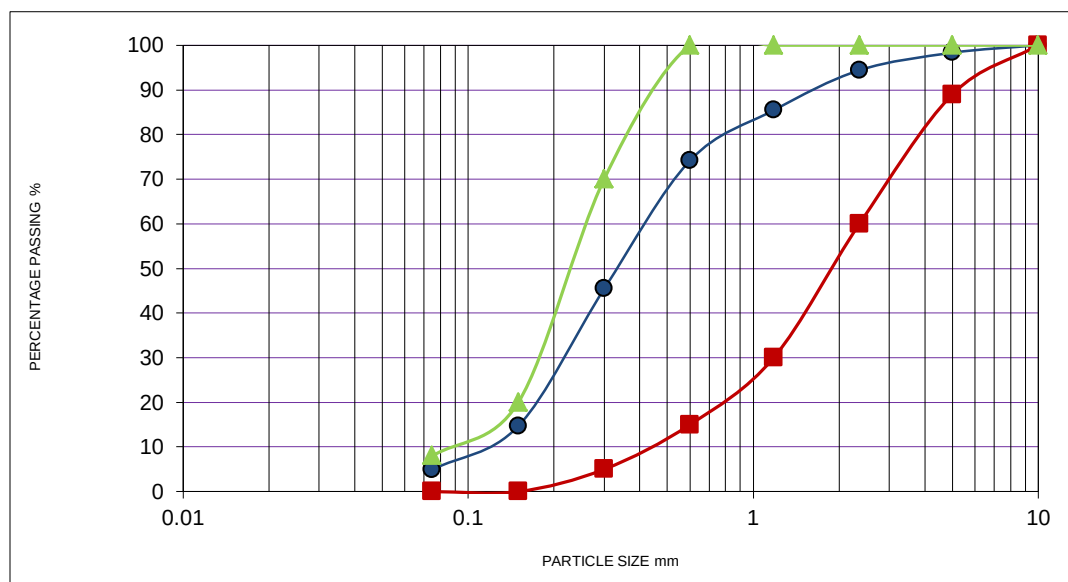


Fig. 4.1 Particle Size Distribution (fine aggregates)

It can be seen from figure 4.1 that, the fine aggregates (sand) is of well graded material. The specific gravity of sand as tested in chapter three is presented in table 3.9 of chapter three and has a value of 2.62. Thus the fine aggregates used in the research work is of well graded sand with specific gravity of 2.62. This value is normal for sand.

(b) Coarse Aggregates

(i) Ten (10mm) Coarse Aggregates

The sieve analysis (particle size distribution) of 10mm coarse aggregates used in this research is presented in table 3.6 of chapter three and the graph is as presented in figure 4.2 below.

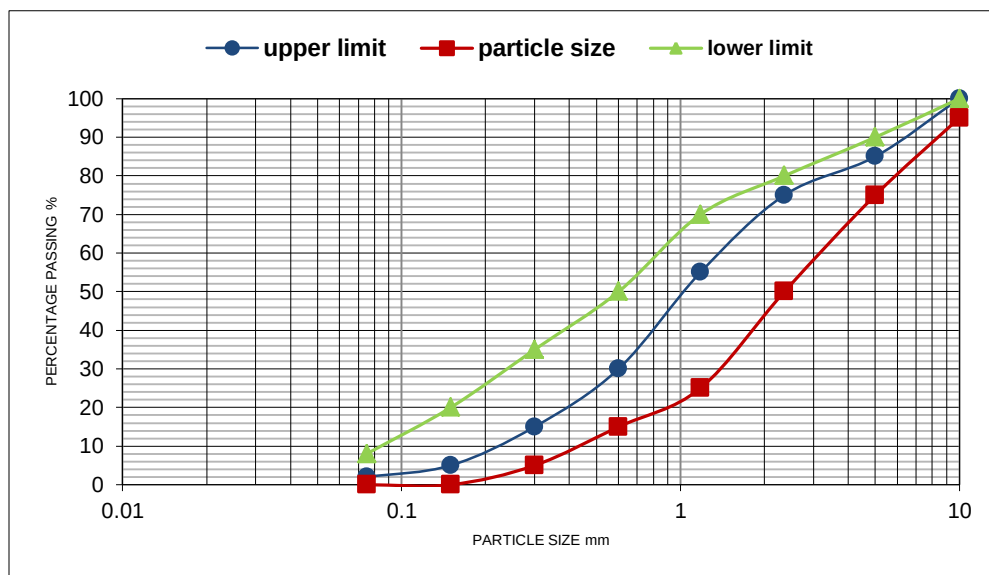


Fig. 4.2 Particle Size Distribution (10mm aggregates)

From the figure, it can be seen that, the coarse aggregates (10mm) used in this research is very good crushed stone as the envelope fall within the specified limit by Code. The steepness of the graph is due to single size aggregates. The specific gravity, aggregates crushing value and water absorption of the aggregates as tested in chapter three are presented in tables 3.9, 3.10 and 3.11 respectively of chapter three. We can therefore say that, the 10mm coarse aggregates used in this

research is of well graded with specific gravity, aggregates crushing value and water absorption of 2.66, 18.7% and 1.43 respectively which are normal for coarse aggregates.

(ii) Fourteen (14mm) Aggregates

The sieve analysis (particle size distribution) of 14mm coarse aggregates used in this research is presented in table 3.7 of chapter three and the graph is as presented in figure 4.3 below.

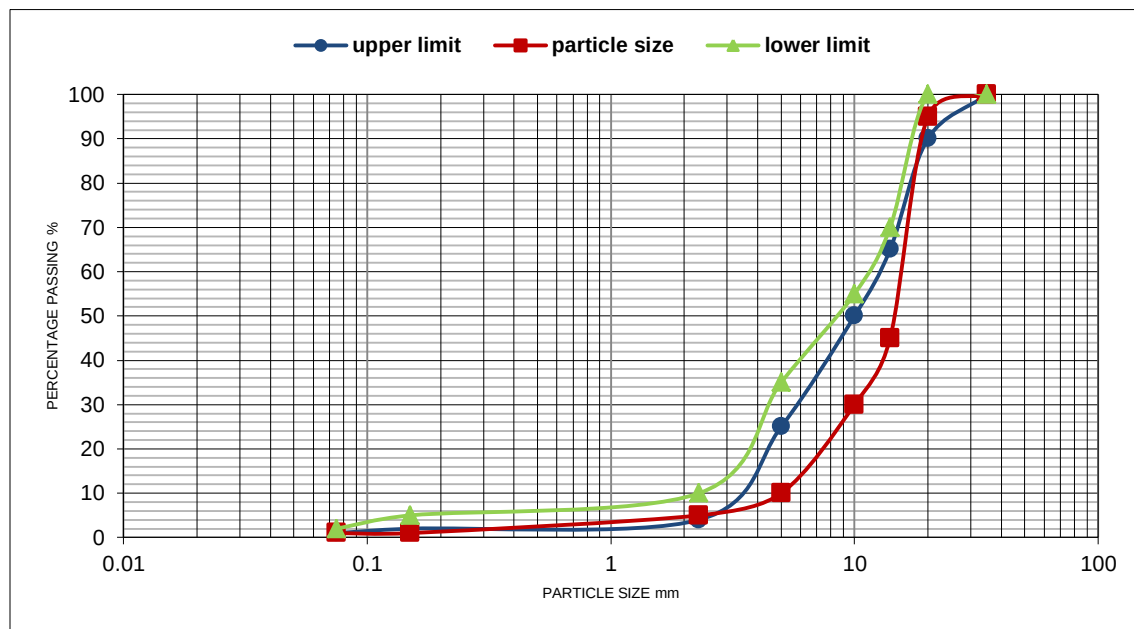


Fig. 4.3 Particle size distribution (14mm aggregates)

It can be seen from figure 4.3 that, the coarse aggregates (14mm) is of good quality material. The specific gravity of aggregate as tested in chapter three is presented in table 3.9 of chapter three and has a value of 2.67. Thus the 14mm aggregates used in the research work is of well graded crushed material with specific gravity of 2.67. This value is normal for coarse aggregates.

(iii) Twenty (20mm) coarse Aggregates

The sieve analysis (particle size distribution) of 20mm coarse aggregates used is presented in table 3.8 of chapter three and the graph is as presented in figure 4.4 below.

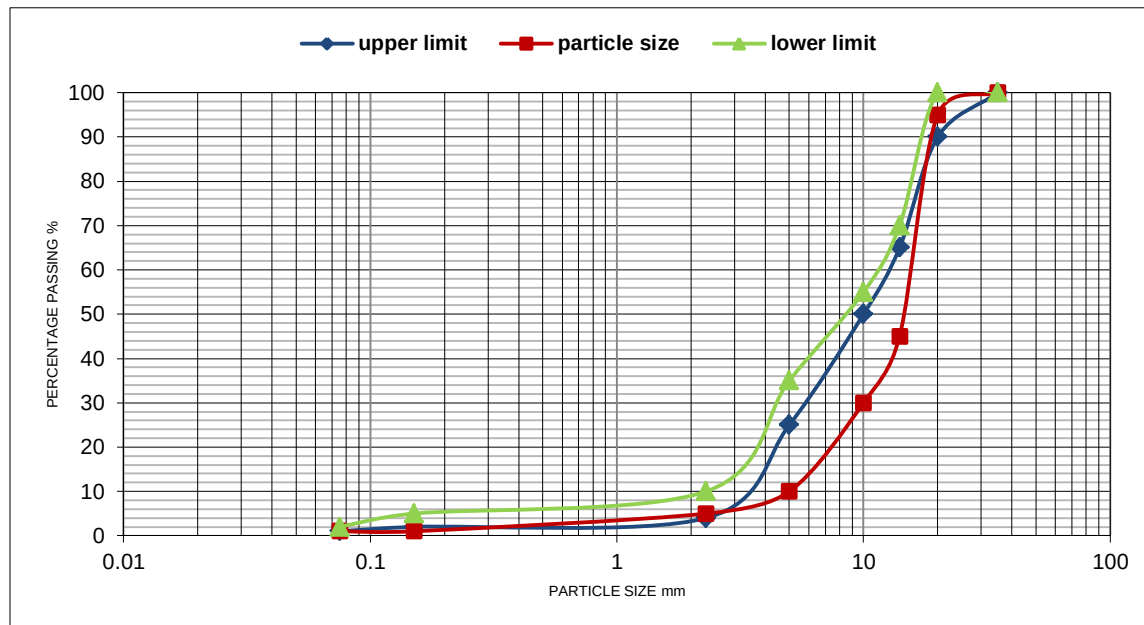


Fig. 4.4 Particle Size Distribution (20mm aggregates)

It can be seen from figure 4.4 that, the (20mm) coarse aggregates is of sound quality material. The specific gravity, water absorption and aggregates crushing values of 2.66, ≤ 2.92 , and 18.7% respectively as presented in table 3.9 of chapter three.

4.3.2 Specific Gravity

The specific gravity of both fine and coarse aggregates used in this research is as presented in table 3.9 of chapter three. The result is compared with standard in table 4.2 below. From the result obtained in chapter three, it is clear that, the specific gravity for both fine and coarse aggregates met the code specified value of less or equal to ≤ 2.92 . Thus, the coarse aggregates used in this research work are of sound quality. The value of the specific gravity obtained for all the aggregates are within the Code range.

Table 4.2: Comparism of Specific gravity with standard value

Samples	Aggregate sizes(mm)	Average G_s grm/m ³	Codes specification BS 1377:-3 1990
A	20	2.660	≤ 2.92
B	14	2.672	
C	10	2.663	
D	Stone dust	2.665	
E	River sand	2.615	

4.3.3 Aggregate Crushing Value

The test result of aggregates crushing value of coarse aggregates used in this research is as presented in table 3.10 of chapter three. The result is also compared with standard in table 4.3 below. From the result obtained in chapter three, the respective average aggregates crushing value for sample A, B, and C. are **17.7%, 20.9% and 17.5%** respectively. The average value for the three samples is (18.7%) which agreed with range of code specified value which is between twelve (12%) and less or equal to twenty three 23% ($12 \leq 23 \leq$ %). This clearly implied that the aggregates crushing values of the tested aggregates are quiet encouraging as they all fall within the range of values specified by the code. They can therefore be used for concrete.

Table 4.3: Comparism of aggregates crushing value with standard value

Test No	Sample A(g)	Sample B(g)	Average	Total average	Specification BS812
1	2419.5	428.7	17.7%	= 18.7%	12-23%
2	2787.3	581.2	20.9%		
3	2754.0	482.3	17.5%		

Where sample A and B denotes combine aggregates and single size aggregates.

. 4.3.4 Water Absorption

The test results of water absorption carried out on coarse aggregates used in this research were presented in table 3.11 of chapter three. The average value of the parameter tested above is also compared to the standards and specification as show in table 4.4 below. From table 4.4, the coarse aggregates parameter tested satisfied to codes of practice. Thus we can say that the aggregates used in this test are of good quality.

Table 4.4: Comparism of water absorption with standard

Size of aggregates	Average value of water absorption%	BS EN 197-1 (1995) and F. M. W. H & T.(1997)
20	1.5	1.25- 2.9%
14	1.4	
10	1.2	
Sharp sand	1.1	
Stondust	1.1	

The water absorption of all aggregates is okay

4.4: Mixing Water

The water used in mixing operation of concrete and cement mortar is portable, thus good for mixing concrete.

4.5: Tests on Fresh Concrete (Workability)

The test results of workability of different concrete mix carried out on this research were presented in table 3.12 of chapter three. The average workability value of some of the mixes is compared with Standard specification as shown in table 4.5 below. From the results, it can be seen that, all the results are consistent with the specification as they fall within the code specified

limit requirements of ASTM 143 and EN 12350-2 1997. It can therefore be said that, all the concrete mixes are of good workability, and within the Slump Limit.

Table 4.5: Comparism of workability with standard specification

Test no	Slumps	Average	Whole average	ASTM 143, EN12350-2. 1997
1	62, 69, 65	65mm	55mm	20 -80 mm
2	64, 67, 68	66mm		
3	57,49, 47	51mm		
4	50, 49, 59	52mm		
5	45, 47, 45	47mm		
6	4.9, 53,49	50mm		

4.6: Percentage Weight Reduction

Fig. 4.5 to 4.24 below give the percentage weight loss of various aggregates concrete mixes of M_1 , M_2 , M_3 and M_4 after exposure of the specimens to different elevated temperatures of 100°C , 300°C , 500°C , 700°C , and 900°C and exposure Time of 30min, 60min., 90min. and 120minutes duration. These values of weight reduction are observed to be varied from 1.6% and 1.1%, to 14.6% to 13.1% corresponding to 1- day age and 28- day age at 30min and 120min duration respectively. The figures mentioned below are arranged according to time of exposure

4.6.(1)- Thirty Minutes (30min) Exposure

The result of percentage weight reduction of concrete cubes specimens subjected to different elevated temperature and maintained for the period of thirty minutes (30min) is presented in table 3.17 to 3. 41 of chapter three and the graphs are as presented in figures 4.5 to 4.9 below.

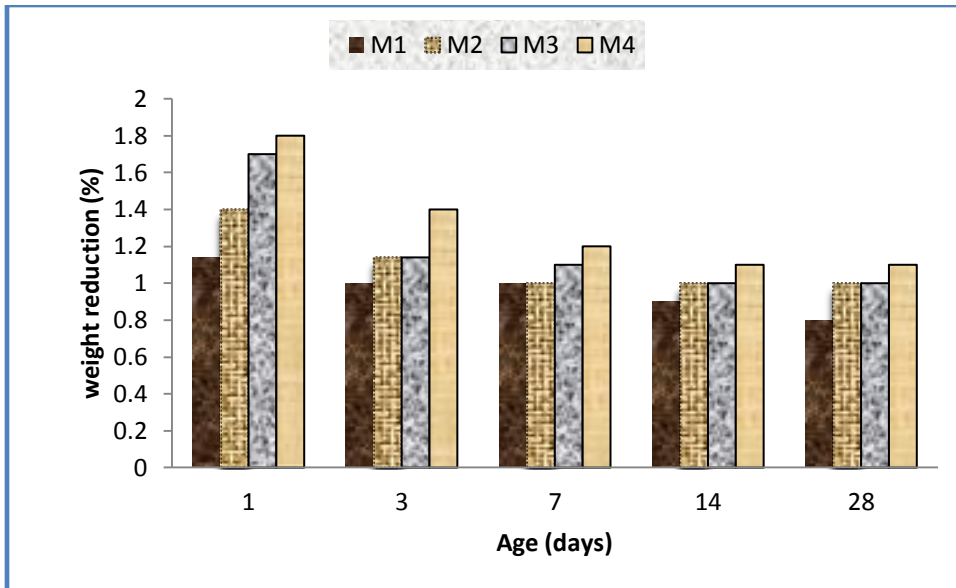


Fig.4.5 Percentage weight reduction at 100^{0c} for 30minutes exposure

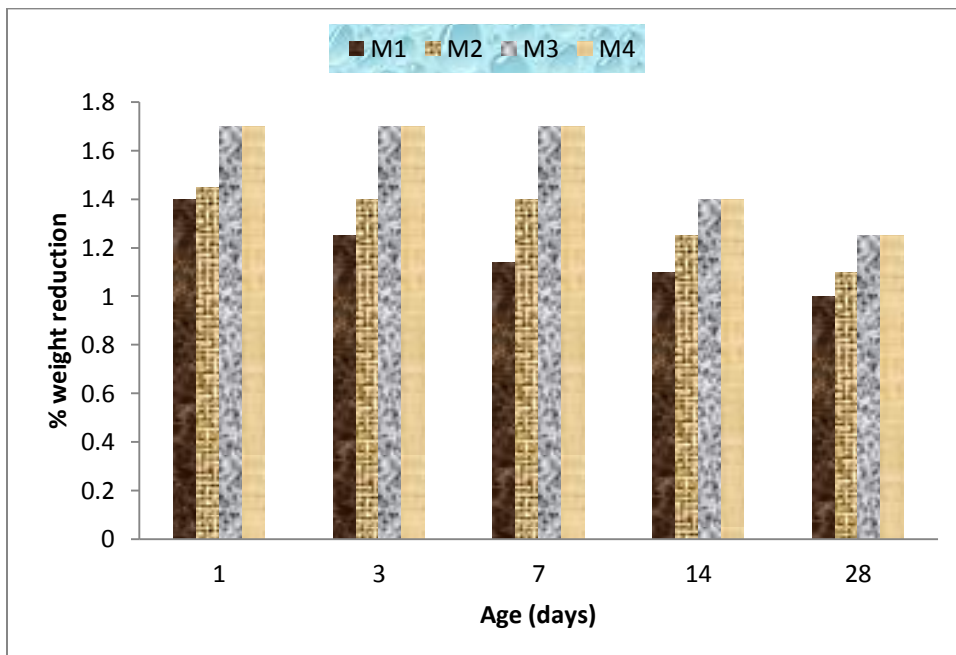


Fig.4.6 Percentage weight reduction at 300^{0c} for 30minutes exposure

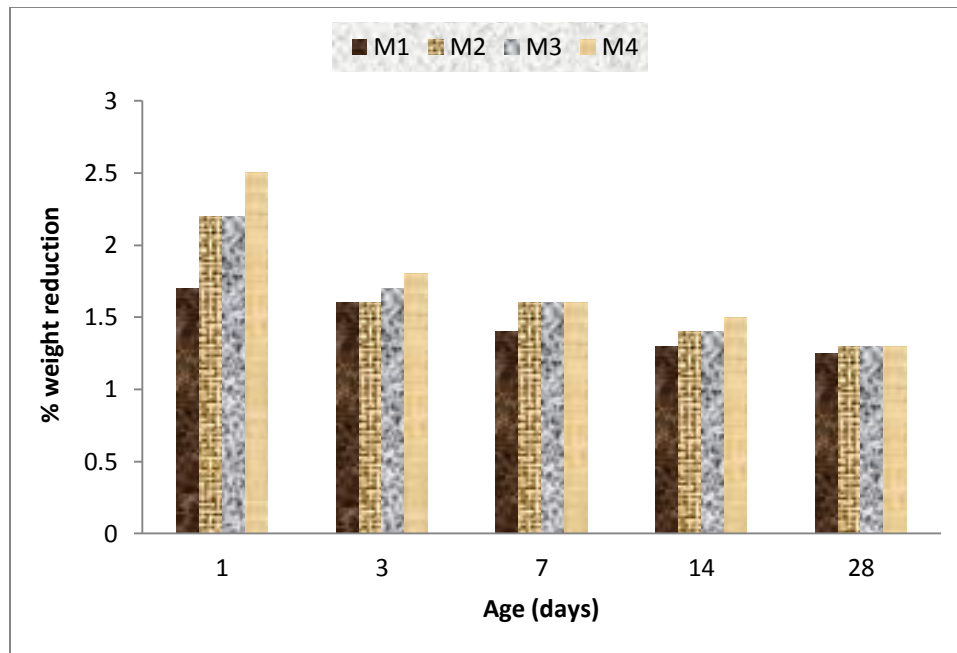


Fig.4.7 Percentage weight reduction at 500^{0c} for 30minutes exposure

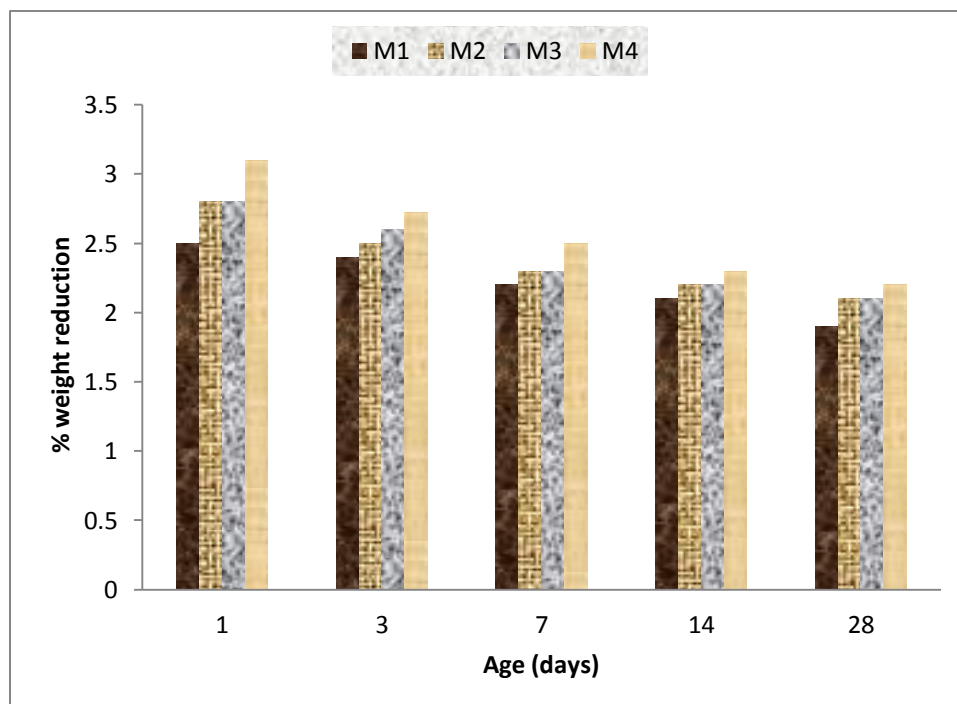


Fig.4.8 Percentage weight reduction at 700^{0c} for 30minutes exposure

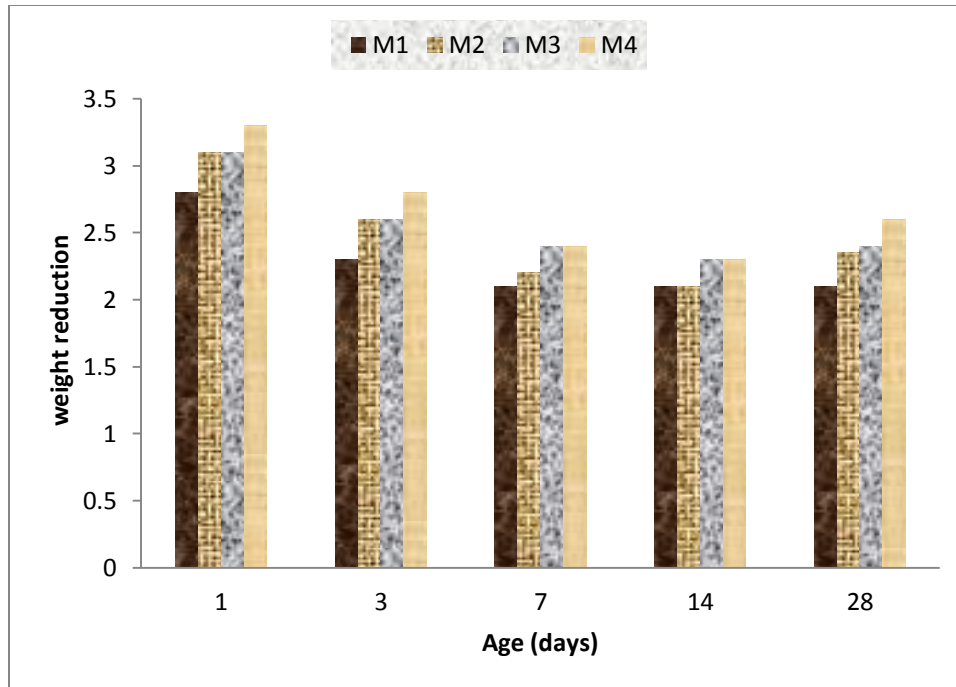


Fig.4.9 Percentage weight reduction at 900^{0c} for 30minutes exposure

The results represent the variation of the percentage weight reduction with temperature for four (4) different concretes mix made from different aggregates sizes labeled M₁,M₂,M₃ and M₄ where M₁,M₂,M₃ and M₄ denote combined aggregates concrete, 10mm aggregates concrete, 14mm aggregates concrete and 20mm aggregates concrete respectively. The concrete specimens were heated to different elevated temperatures and were maintained at those temperatures for the period of thirty minutes exposure as earlier stated. From the results, it can be observed that, the percentage weight reduction increases with increase in temperatures and reduces as concrete ages increases. Also the result show that combined aggregates concrete appeared to have better fire performance than concrete made using single size aggregates as it shows less weight loss in all cases. The effect of concrete age is displayed in all figures that there is more weight reduction at 1-day age concrete than at 28-day age in all cases. For example, in figure 4.5, the respective average weight loss at 100^{0C} for 1 and 28 day age is 1.3% and 0.9% at 30minutes exposure.

Table 4.6 summary of percentage weight reduction

Temperature (⁰ C)	Exposure time (min)	Concrete age (days)	Average%Weight lost
100 ⁰ C	30min	1-day age	1.3%
		28-day age	0.9%
500 ⁰ C		1-day age	1.7%
		28-day age	1.3%
900 ⁰ C		1-day age	3.0%
		28-day age	2.4%.

However, when the temperature was raised to 500 $^{\circ}\text{C}$ as displayed in table 4.6, the average weight lost recorded is 1.7% and 1.3% for 1 and 28 day age respectively. When the exposure temperature was further raised to 900 $^{\circ}\text{C}$ as in table 4.6, the respective weight lost recorded for 1 day and 28 days age is amounting to 3% and 2.4%. The result generally show that, combined aggregates concrete has better fire performance than the concrete made using single aggregates size. In the case of concrete made using single size aggregates, M₂ performs better than M₄. This shows that, as the aggregates increases in size, more void will be created and loss in weight will be increased. The same trend is displayed in all the remaining figures that M₁ displayed the least reduction in weight than the rest Ms in all cases due to fewer voids. Furthermore, in all cases, the highest weight reduction is recorded at the age of 1 day particularly in respect of 900 $^{\circ}\text{C}$ exposure

temperature. The same phenomenon had also been recorded by Ahmed and Fahmi (2004). This was attributed to the fact that, at early age there was more moisture to be evaporated than at later age.

(ii) Sixty Minutes (60min) Exposure

The result of percentage weight reduction of concrete cubes specimens subjected to different elevated temperature and maintained for the period of Sixty minutes exposure (60min) is presented in table 3.42 to 3. 66 of chapter three and the graph are as presented in figures 4.10 to 4.14 below.

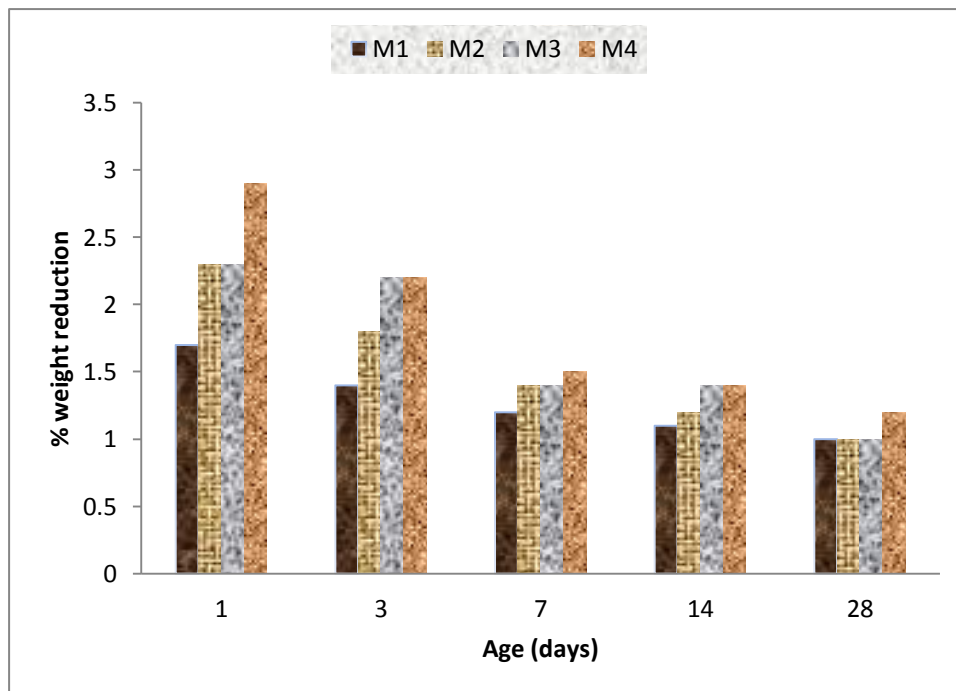


Fig.4.10 Percentage weight reduction at 100^{0c} for 60minutes exposure

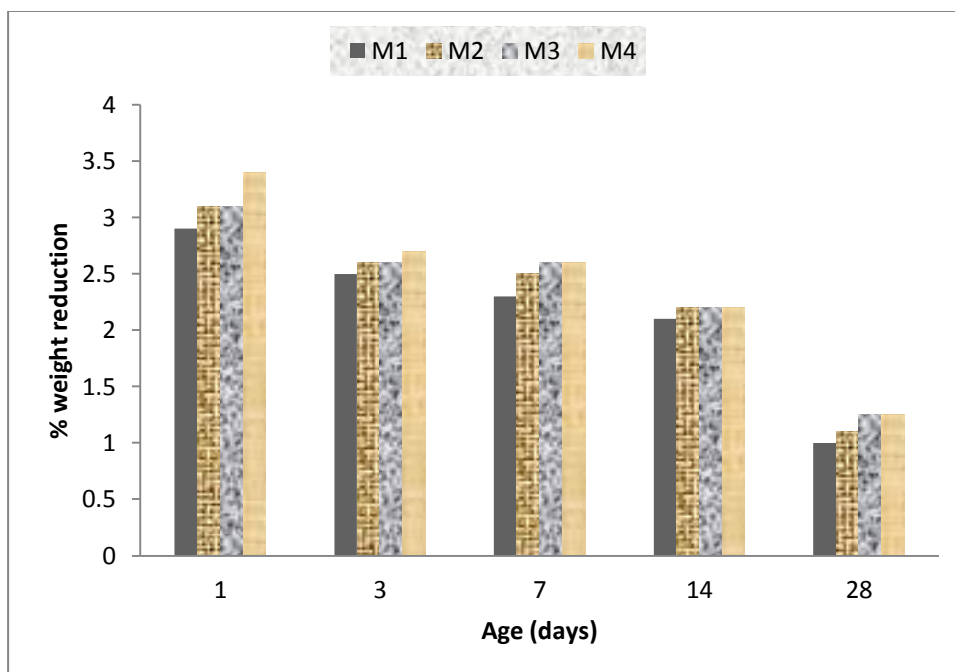


Fig.4.11 Percentage weight reduction at 300^{0c} for 60minutes exposure

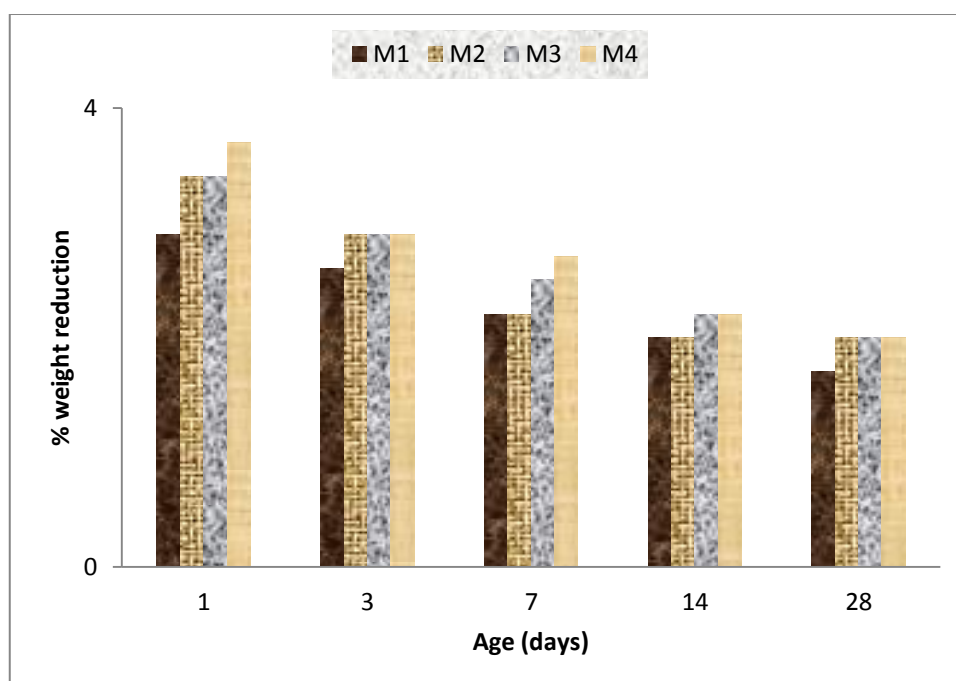


Fig.4.12 Percentage weight reduction at 500^{0c} for 60minutes exposure

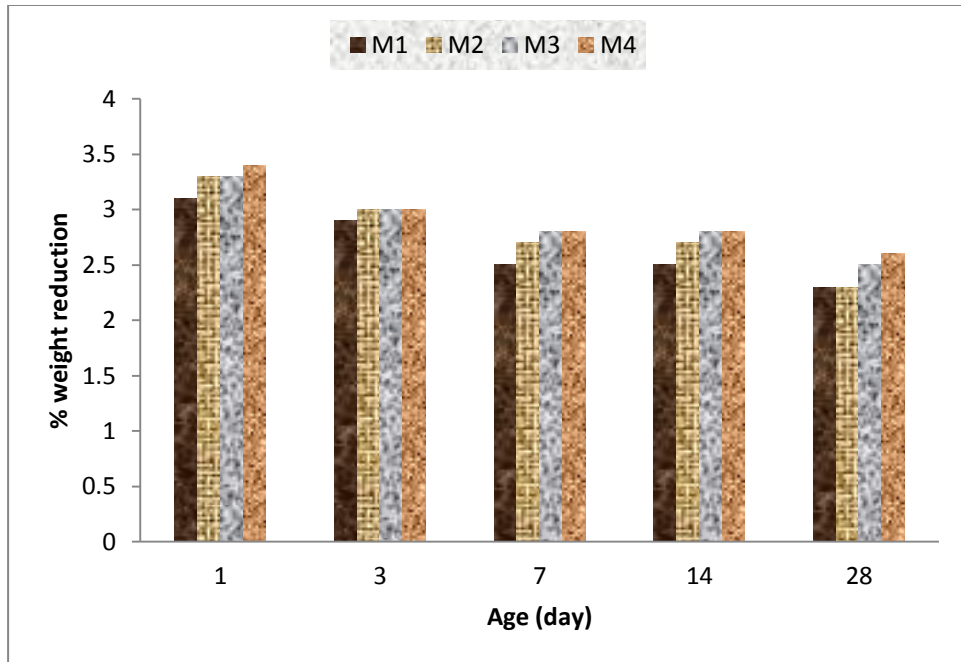


Fig.4.13 Percentage weight reduction at 700^{0c} for 60minutes exposure

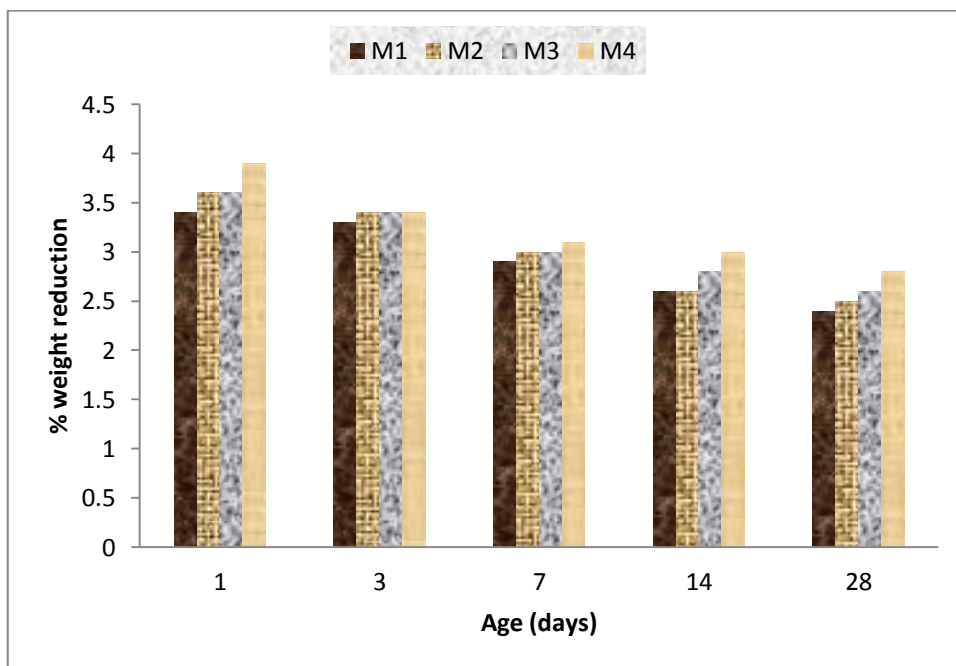


Fig.4.14 Percentage weight reduction at 900^{0c} for 60minutes exposure

As earlier stated, the results represent the variation of the percentage weight reduction with temperature for four (4) different concretes mix made from different aggregates sizes labeled M_1 , M_2 , M_3 and M_4 . The concrete specimens were kept for the period of sixty minutes under different elevated temperatures. The result shows significant increase in weight lost as a result of increase in exposure time. The usual trend is that, the percentage weight reduction increases with increase in temperatures and reduces as concrete attains maturity age. Just like in thirty minutes exposure, when the time is further raised to sixty minutes, M_1 concrete still appeared to have less weight lost compare to M_2 , M_3 and M_4 and M_4 concrete yields larger weight lost in all cases. A 1- day age and 28- day age concrete was considered to demonstrate the effects of both concrete age and exposure period on residual weight lost. From figure 4.10 and 4.14, the average weight lost at both 100°C and 900°C for 60minutes exposure is amounting to 2.3% and 1.1% at and 3.7% and 2.8% respectively. When above mentioned result were compared to that in figure 4.5 and 4.9 at 30minutes exposure, it can be seen that, there is a difference of about 1% and 2% increase respectively. In all figures there is more weight reduction at 1-day age concrete than at 28-day age in all cases. For instance, in figure 4.5, the respective average weight loss for 1-day and 28 day age is 1.3% and 0.9% at 30minutes exposure. Generally the average weight loss is reduced at higher age when compared to lower age. The discussed result is now summarized in table 4.7 below

Table 4.7 summary of percentage weight reduction

Temperature (°C)	Exposure time (min)	Concrete age (days)	Average %Weight reduction
100 ⁰ C	60min	1-day age	2.3%
		28-day age	1.1%
500 ⁰ C		1-day age	3.1%
		28-day age	1.4%
900 ⁰ C		1-day age	3.7%
		28-day age	2.8%

(iii) Ninety Minutes (90min) Exposure

The result of percentage weight reduction of concrete cubes specimens subjected to different elevated temperature and maintained for the period of ninety minutes exposure (90min) is presented in table 3.67 to 3. 91 of chapter three and the result are plotted in figures 4.15 to 4.19 below.

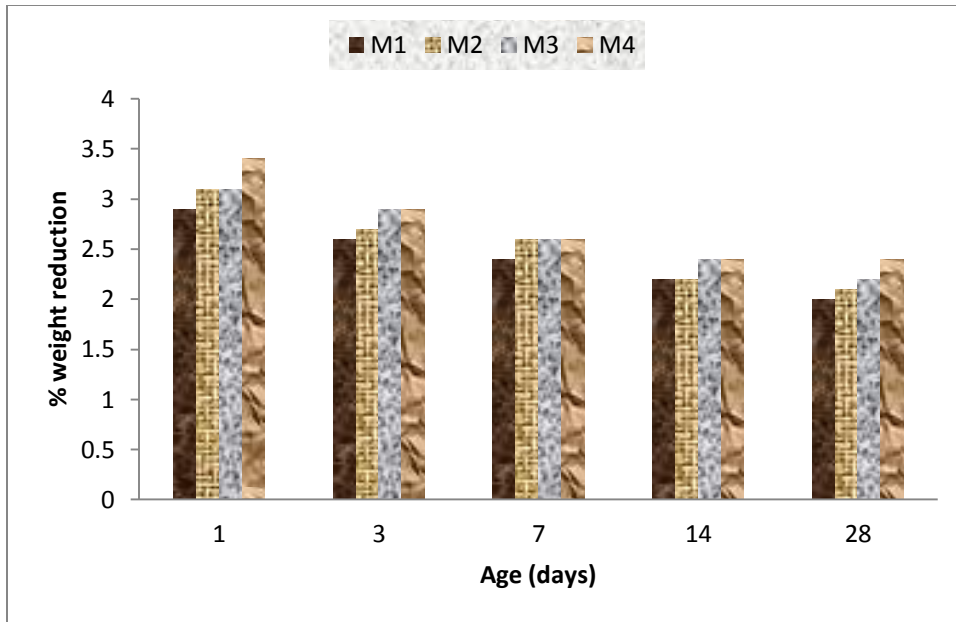


Fig.4.15 Percentage weight reduction at 100^{0c} for 90minutes exposure

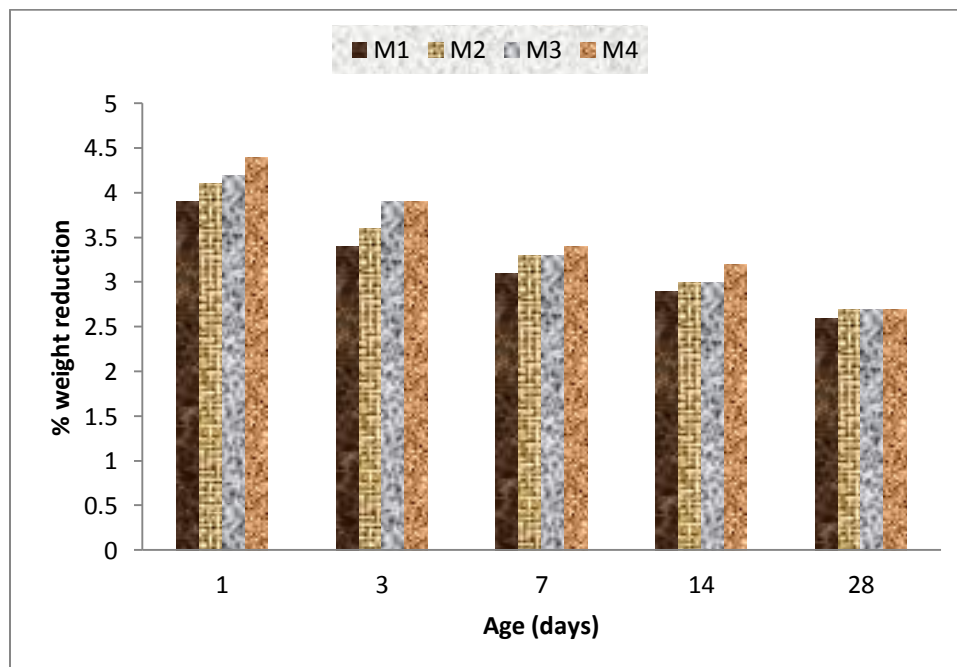


Fig.4.16 Percentage weight reduction at 300^{0c} for 90minutes exposure

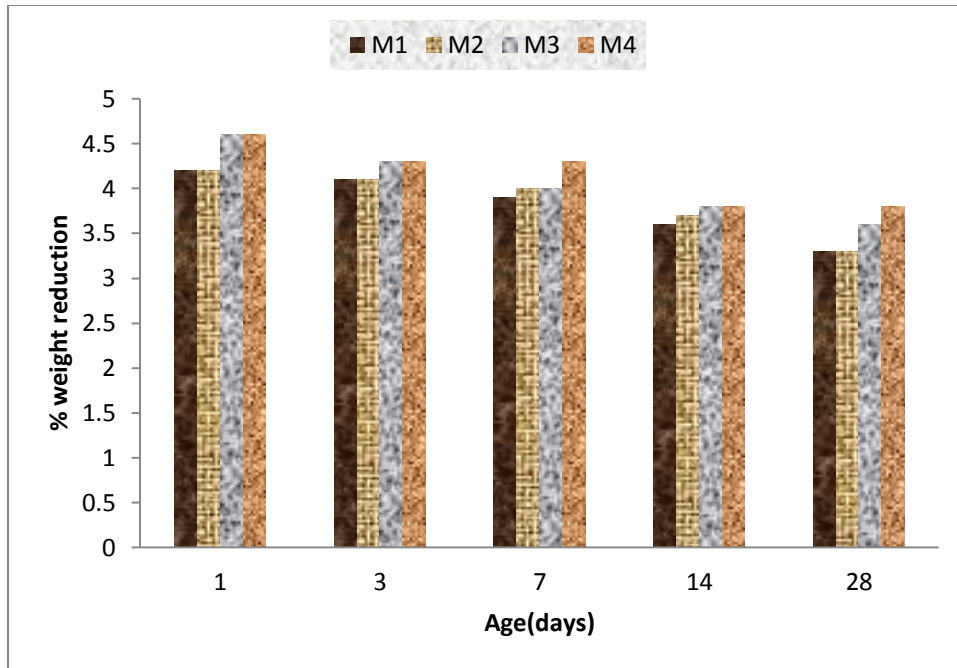


Fig.4.17 Percentage weight reduction at 500^{0c} for 90minutes exposure

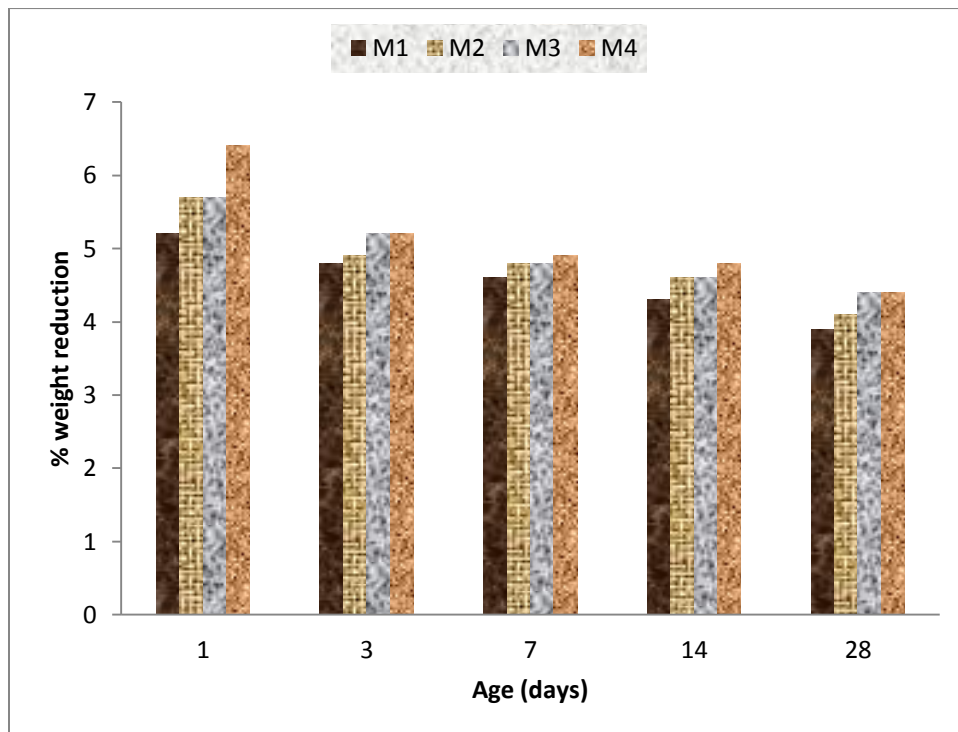


Fig.4.18 Percentage weight reduction at 700^{0c} for 90minutes exposure

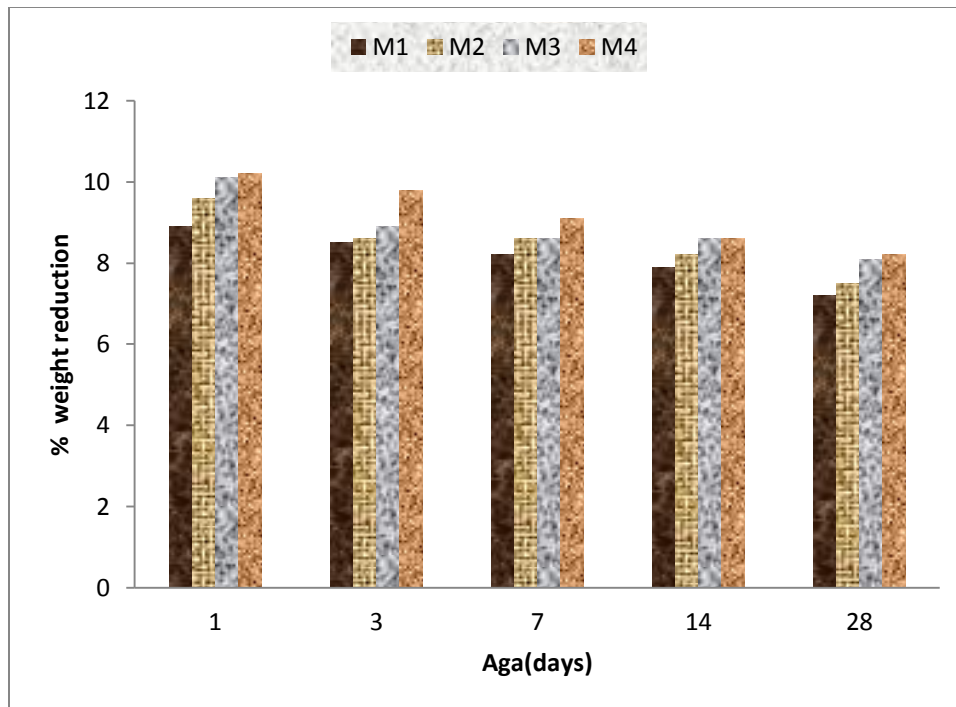


Fig.4.19 Percentage weight reduction at 900⁰c for 90minutes exposure

The results represent the variation of the percentage weight reduction with temperature at ninety minutes exposure for four (4) different concretes mix as earlier stated. As a result of increase in exposure time to 90minutes, the result shows significant increase in weight lost amounting to 3.0% and 2.4% at 100⁰C and 4.3% to 3.4% at 500⁰c corresponding to 1 and 28-days age concrete. At maximum temperature (900⁰c), 10.1% and 8.2% was recorded all corresponding to 1-day age and 28- day age concrete respectively. The usual trend is also generally maintained that, combined aggregates concrete (M₁) still appeared to have less weight lost due minimum voids as different aggregates sizes form dense concrete because of mechanical keying among various shapes of aggregates compare to single size aggregates concretes (M₂, M₃, and M₄) and that M₄ concrete yields larger weight lost in all cases. Also the percentage weight reduction increases with increase in temperatures and reduces as concrete attains maturity age. This was mainly attributed to the accelerated hydration of cement, as well as the drying with increased in

temperature due to high moisture content at early age than at later age. The same phenomenon had also been reported by Fahmi and Mohamedbhai. See table below. The summary of the obtained result from the discussion is summarized in table 4.8 below.

Table 4.8 summary of percentage weight reduction

Temperature (⁰ C)	Exposure time (min)	Concrete age (days)	Average %Weight reduction
100 ⁰ C	90min	1-day age	3.0%
		28-day age	2.4%
500 ⁰ C		1-day age	4.3%
		28-day age	3.4%
900 ⁰ C		1-day age	10.1%
		28-day age	8.2%

(iv) One Hundred and Twenty Minutes (120min) Exposure

The result of percentage weight reduction of concrete cubes specimens subjected to different elevated temperature and maintained for the period of One Hundred and twenty minutes exposure (120min) is presented in table 3.92 to 3. 116 of chapter three and the result are plotted in figures 4.20 to 4.24 below.

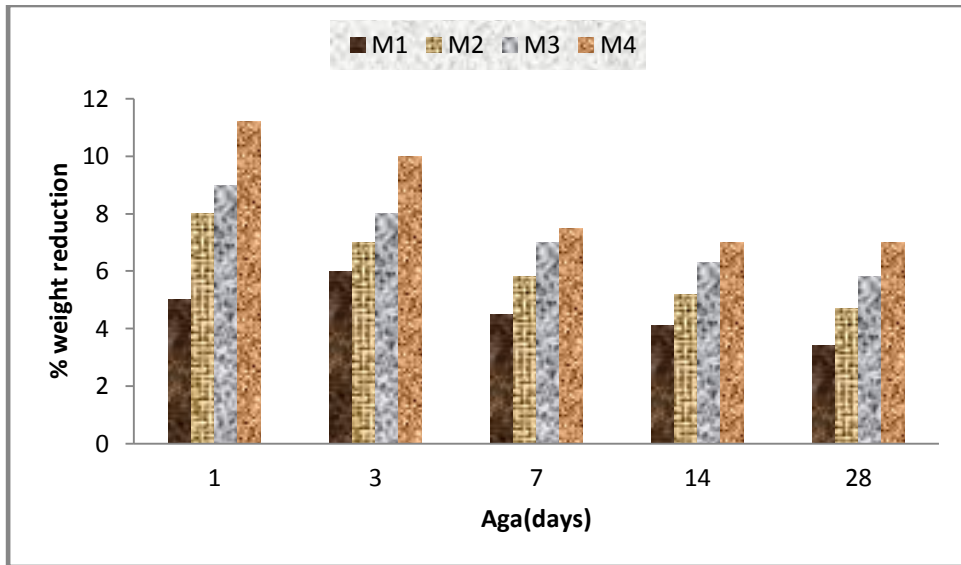


Fig.4.20 Percentage weight reduction at 100^oC for 120minutes exposure

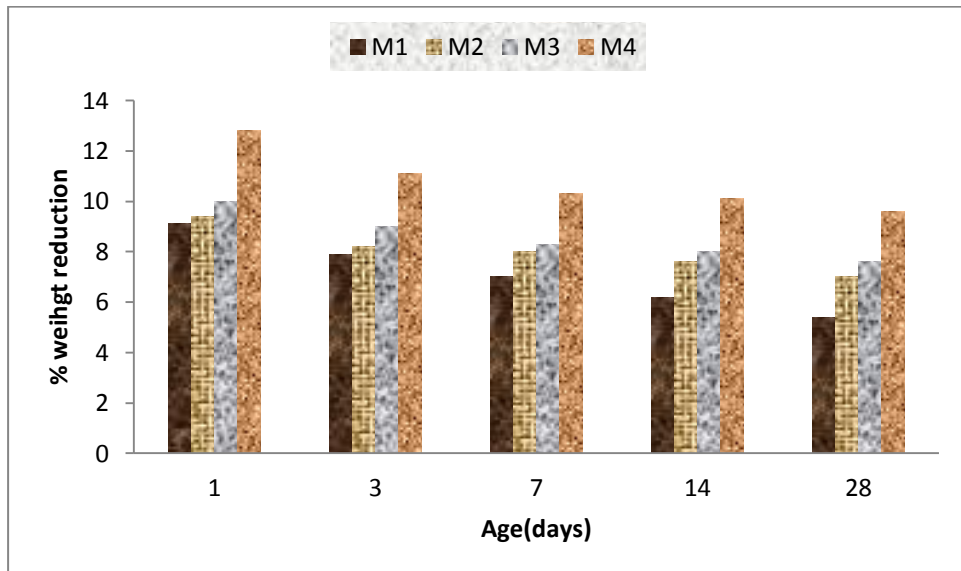


Fig.4.21 Percentage weight reduction at 300^oC for 120minutes exposure

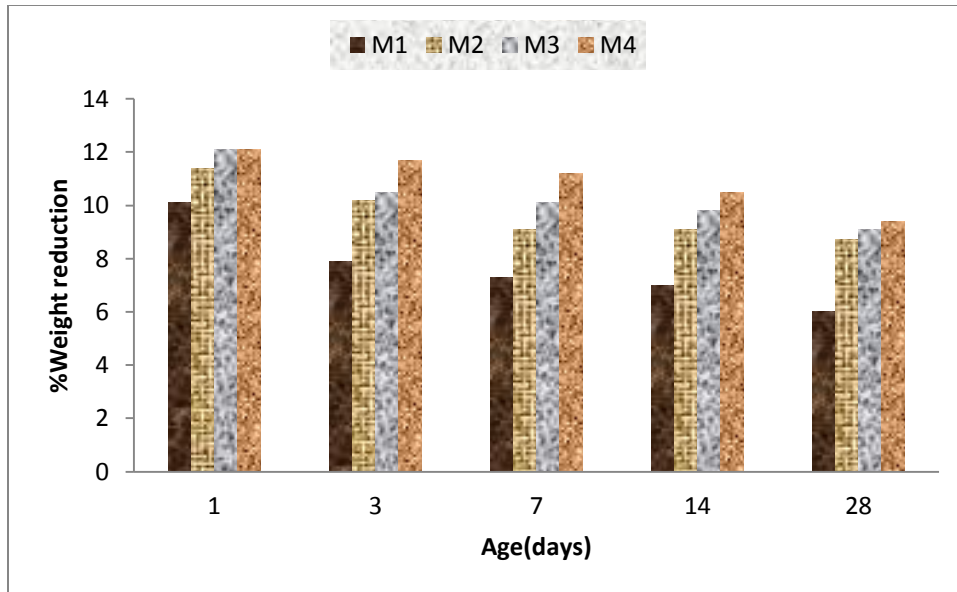


Fig.4.22 Percentage weight reduction at 500^{0c} for 120minutes exposure

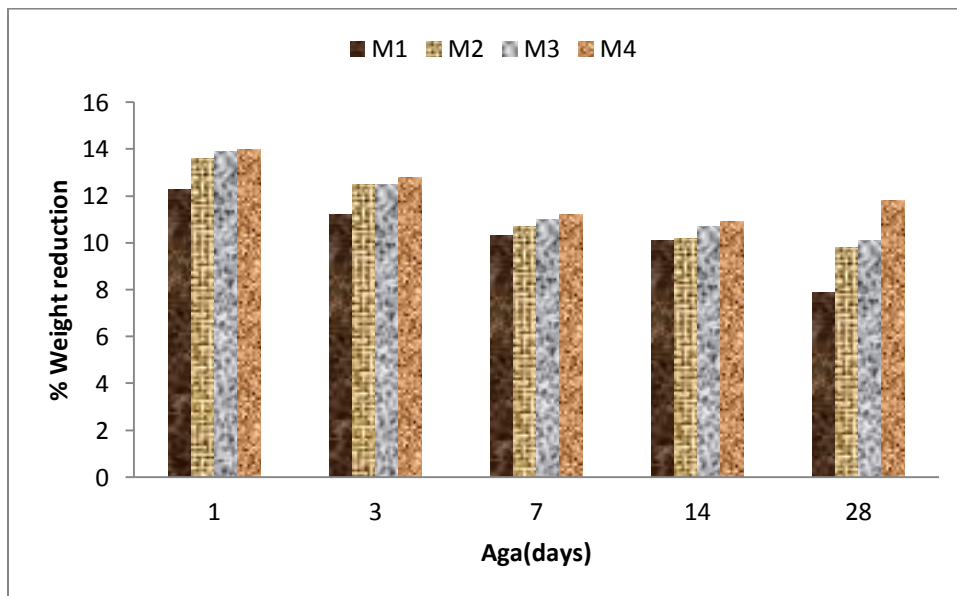


Fig.4.23 Percentage weight reduction at 700^{0c} for 120minutes exposure

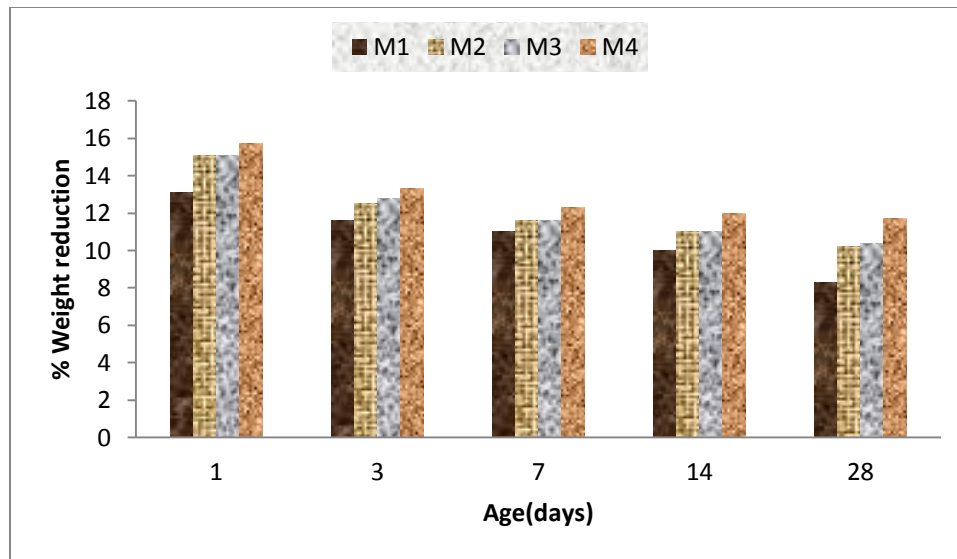


Fig.4.24 Percentage weight reduction at 900^{0c} for 120minutes exposure

The results represent the variation of the percentage weight reduction with temperature at one hundred and twenty minutes exposure for four (4) different concretes mix as earlier stated. As a result of increase in exposure time to 120minutes, the result shows significant increase in weight loss amounting to 14.2% and 14.6%, at 900^{0C}. The weight loss recorded at 500^{0C} is 10.5% and 7.4%, while the recorded weight loss at 100^{0C} is 9.6% and 4.5% at 120minutes all corresponding to 1- day age and 28- day age concrete respectively. The usual trend is also generally maintained that, combined aggregates concrete (M₁) still appeared to have minimum weight loss due minimum voids as different aggregates sizes form dense concrete due to mechanical keying among various shapes of aggregates compare to single size aggregates concretes (M₂, M₃, and M₄) but M₄ shows more weight loss in all cases. Also the percentage weight reduction increases with increase in temperatures and reduces as concrete attains maturity age. This was mainly attributed to the accelerated hydration of cement, as well as the drying with increased in temperature due to high moisture content at early age than at later age. The same phenomenon

had also been reported by Fahmi and Mohammedbhai. See table 4.9 below for the summary of the analysis

Table 4.9 summery of percentage weight reduction

Temperature (°C)	Exposure time (min)	Concrete age (days)	Average %Weight reduction
100 ⁰ C	120min	1-day age	8.2%
		28-day age	5.5%
500 ⁰ C		1-day age	10.5%
		28-day age	7.4%
900 ⁰ C		1-day age	14.1%
		28-day age	11.0%

Generally, it is evident that

- 1- Combined aggregates concrete appeared to lost minimum weight compared to single size aggregates concrete. This was attributed to the fact that, combined aggregate concrete is denser due mechanical keying and minimum voids compare to single aggregates concrete.
- 2- There was significant increase in weight loss as exposure period increased above 60min was progressively increased (from 30minutes to 120minutes respectively).

- 3- That early age concrete had experienced larger amount of weight loss
- 4- Progressive increase in weight lost was also observed when the heating temperature and exposure times were raised from minimum to maximum (100°C to 900°C and 30min to 120min. respectively).
- 5- The average weight loss is reduced at higher age when compared to lower age.

4.7 Visual Inspection

According to Schneider, the assessment of fire- damaged concrete usually starts with visual observation of colour change, cracking and spalling of concrete surface. The surface textures, cracks and spalled section of concrete specimens exposed to high temperatures are illustrated in plate xiv to xix below. It can be observed that there was no visible effect on the specimen's surface heated to 300°C . The crack started to become visible on concrete specimens only when the temperature reached to 600°C , but the effect was not significant at that temperature level. The effect was more pronounced when the temperature was raised to 900°C and above where the specimens spalled.

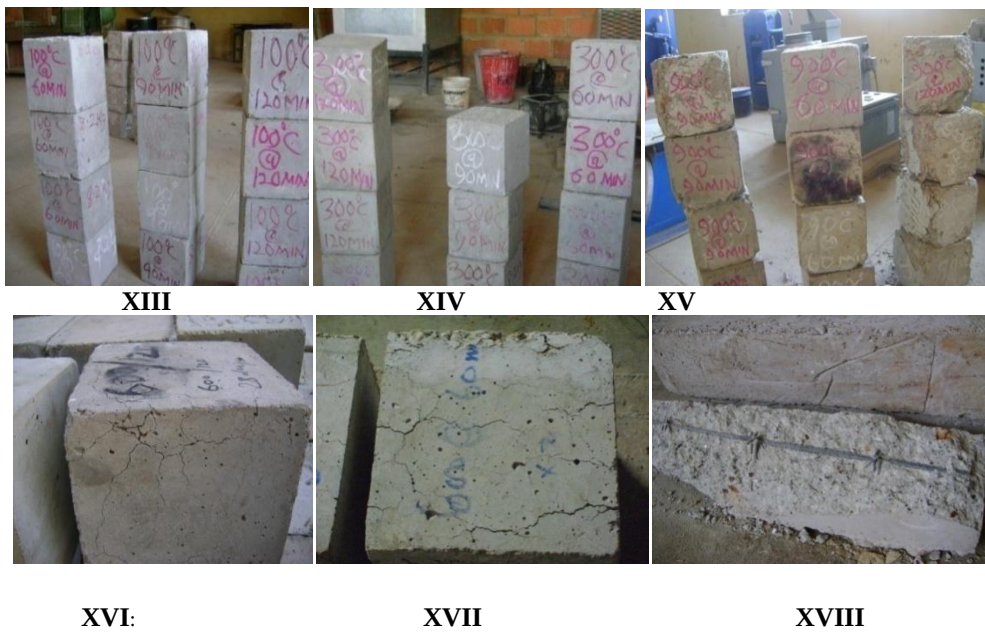


Plate XIII to XVIII show surface textures, colour, cracks and spalling of concrete specimens

4.8 Retained Compressive Strength

In the results, the effects of exposure time, exposure temperature, concrete age and Concrete mix on the residual compressive strength of concrete are discussed.

The strength of concrete is one of the main properties of all concrete structures that resist applied loads (Orio 2001). It is observed that one of the main effects of high temperatures on concrete structures was the reduction in compressive strength. Schneider, (1988), Hertz (2003), Bakharev, (2002), and Collins et al, (2001). The test results revealed that heating concrete cubes specimens to 100⁰C resulted in noticeable increase in compressive strength amounting to an average of 15% after which the strength began to decline progressively with increase in exposure temperature and exposure times. This was attributed to accelerated hydration and drying of specimens when exposed to elevated temperatures. (Ahmed et al, 2004). On the other hand, aggregates size and maturity age significantly contribute to the high performance of concrete at elevated temperatures.

4.8.(i) (1 day age Concrete)

The retained compressive strength values of 1 day age concrete specimens after exposure to high temperatures are presented in table 3.117, 3.122, 3.127 and 3.132 of chapter three. The specimens are made using different aggregate sizes of 10mm, 14mm and 20mm aggregates. The compressive strength was calculated as the percent compressive strength of heated concrete after exposure temperature with respect to strength of unheated specimens.

The results presented on figures 4.31 to 4.34 represent a 1- day age concrete exposed to different elevated temperature and maintained at those temperatures for four different exposure times as earlier said. The results revealed that heating the specimens to 100⁰C resulted in a noticeable

increase in strength amounting to an average of 20% of the original strength of all the concretes. This increase was mainly attributed to the accelerated hydration of cement, as well as the drying with increase temperature. Ahmed et al, (2004). It can be observed from the results that, the increase in exposure time resulted in a steady decrease in strength. For example, the retained compressive strength value at 30minutes (figure 4.31) was 75% at 300°C. The strength now became 48% at the same 300°C when the exposure time was further raised to 120minutes (fig. 4.34) respectively. It is evident that, raising both the temperature and exposure time resulted in further decrease in strength. For example, the retained compressive strength values recorded in fig. 4. 32 were 110% at 100°C and dramatically reduced to almost 12% when the temperature was further raised to 500°C. The specimens finally crumbled to zero% strength at 900°C in all cases. Among all the concretes, M₁ appeared to have high fire performance compared to rest Ms in all figures. From the plots, it may be observed that, the effect of exposure time was more noticeable at high temperatures (500-900°C).

Generally, the specimens suffered a sharp declining of strength when the exposure temperature was raised to 300°C upwards.

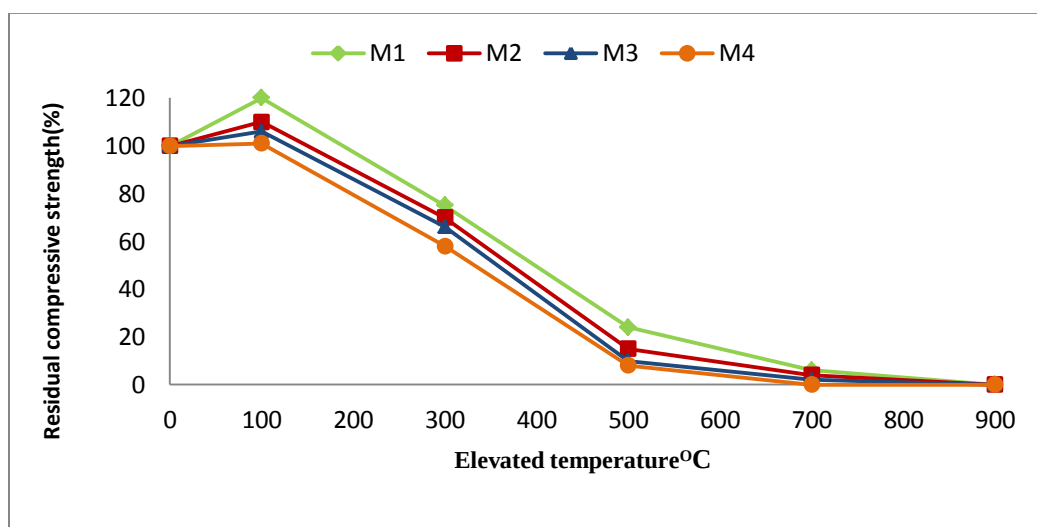


Fig. 4.25 Compressive strength of 1 day age concrete @ 30mins exposure time

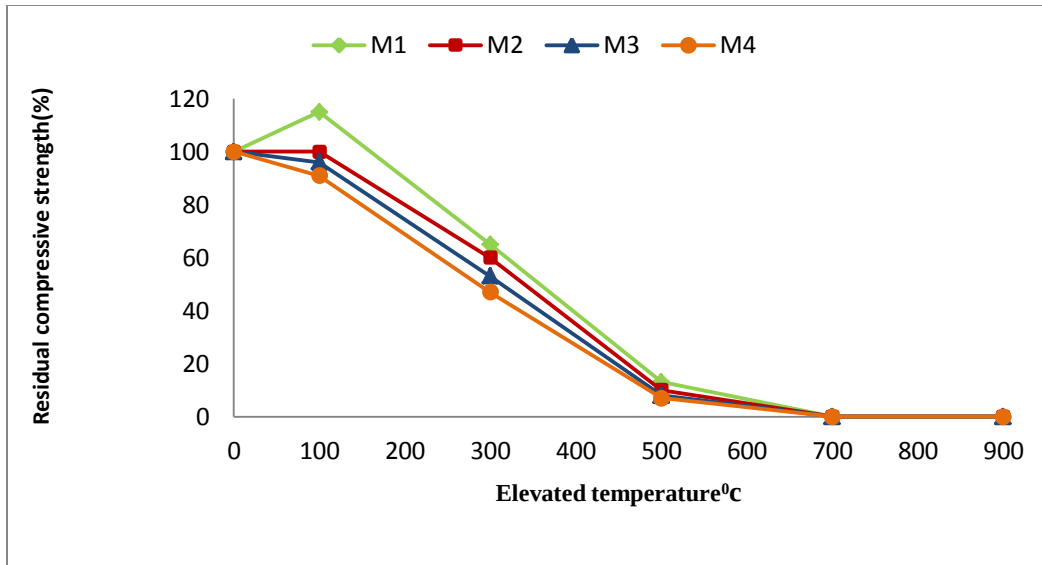


Fig.4.26 Compressive strength of 1-day age concrete @ 60mins exposure time

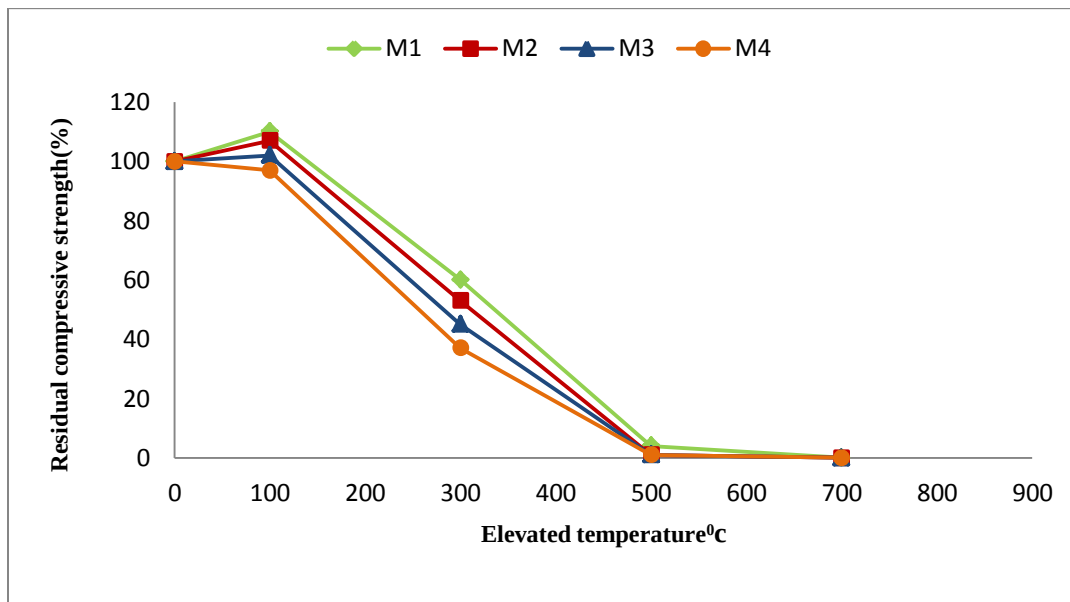


Fig.4.27: Compressive strength of 1-day age concrete @ 90mins exposure time

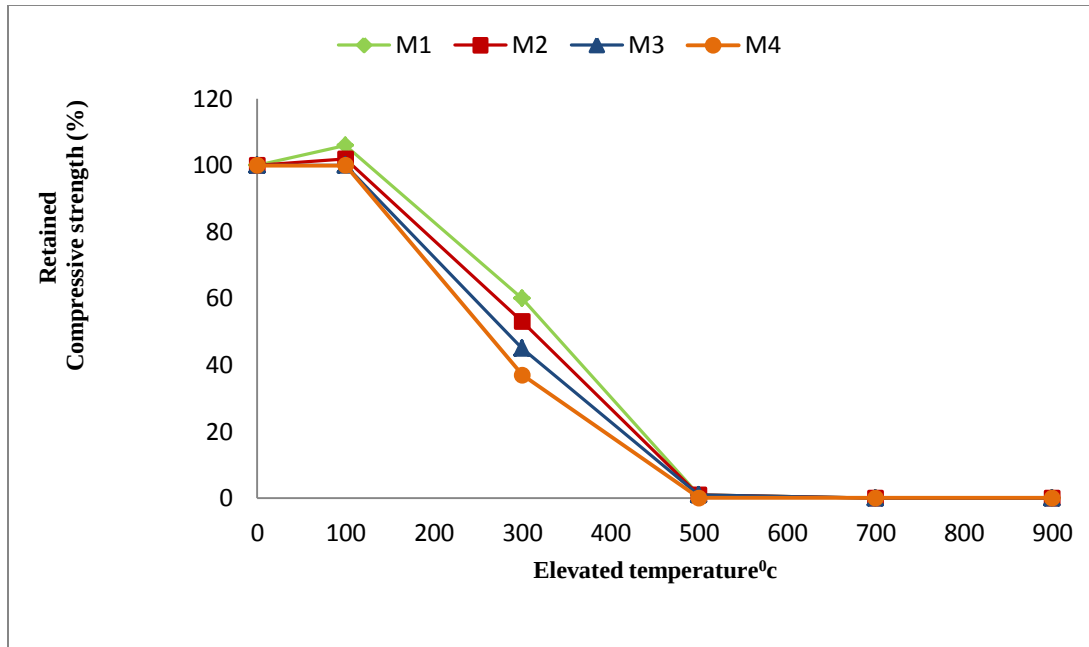


Fig.4.28: Compressive strength of 1-day age concrete @ 120mins exposure time

(ii) (3 day age Concrete)

The retained compressive strength values of 3 day age concrete test specimens after exposure to high temperature are presented in table 3.118, 3.123, 3.128 and 3.133 of chapter three. The retained strengths represent different aggregates concretes labeled M₁, M₂, M₃ and M₄ which employ combined aggregates concrete, 10mm aggregates concrete, 14mm aggregates concrete and 20mm aggregates concrete as earlier said. The retained compressive strength values are shown on figures 4.35 to 4.38. The retained compressive strength values significantly increased at exposure temperature of 100°C. However, the retained compressive strength slightly decreases upon heating to 200°C and then sharply reduced beyond that point. A similar pattern as 1 day age concrete was displayed, that heating to 100°C resulted in a noticeable increase in strength amounting to 18% and dropped to 74% of its original strength when the temperature was further raised to 300°C. The lost of strength associated with increase in temperature in this

range could be as a result of the lost of moisture driven off during heating and incompatibility in thermal expansion between cement paste and aggregates which also accompanied by loss of weight as earlier discussed. (Ahmed et, al, 2004). reported that, the difference between the coefficient of thermal expansion of the cement paste and that of limestone aggregates resulted in thermal incompatibility as a result of which thermal stresses were induced in the paste and aggregates. The induced stresses then resulted in the breakdown of the interfacial bond between the aggregates and the surrounding paste, with consequent strength loss of the concrete. The average percentage reduction in compressive strength recorded when the temperature was further raised to 500⁰C and at 120minutes exposure was 13.3% of the original compressive strength (see fig. 4.38). This was probably due to the continuation of the evolution of water. It may also be attributed to the densification process of the cement gel due to the loss of physically adsorbed water.

Raising exposure temperature and time to 600⁰C and 90minutes caused a continuation in strength loss, with concrete retaining between 10% and 35% of its original strength respectively. At this temperature hair cracks were clearly visible on the specimen's surface as shown in plate below. This could be a result of the evolution of the combined water left, which evaporated at 500⁰C as reported by Lea, 2009. In all cases, M₁ has better fire performance and M₄ appeared to have the least.

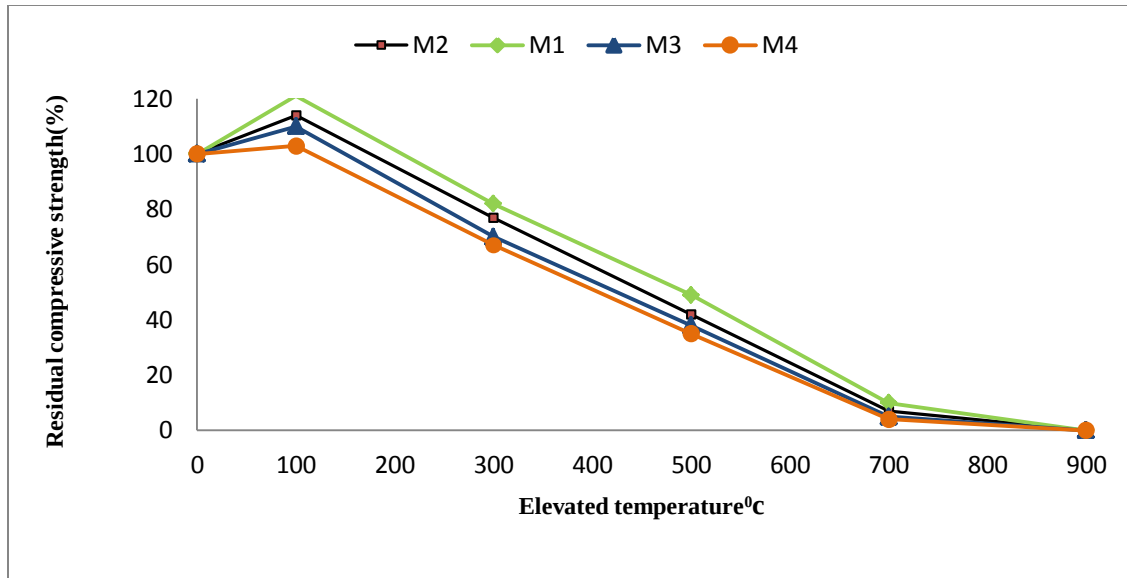


Fig.4.29 Retained compressive strength of 3-days old concrete @ 30mins exposure time.

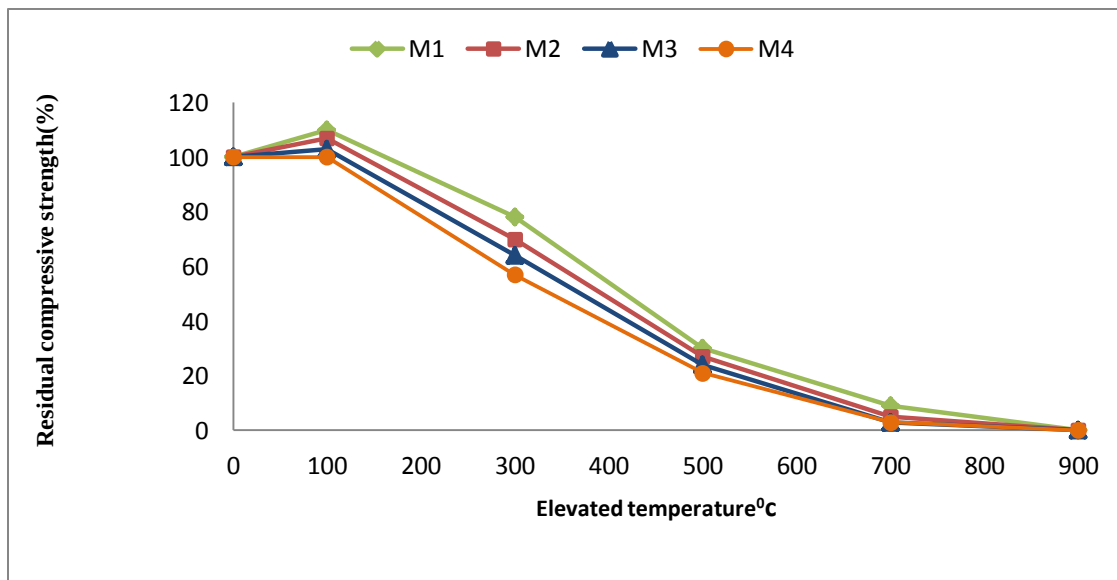


Fig.4.30 Retained compressive strength of 3-days old concrete @ 60mins exposure time

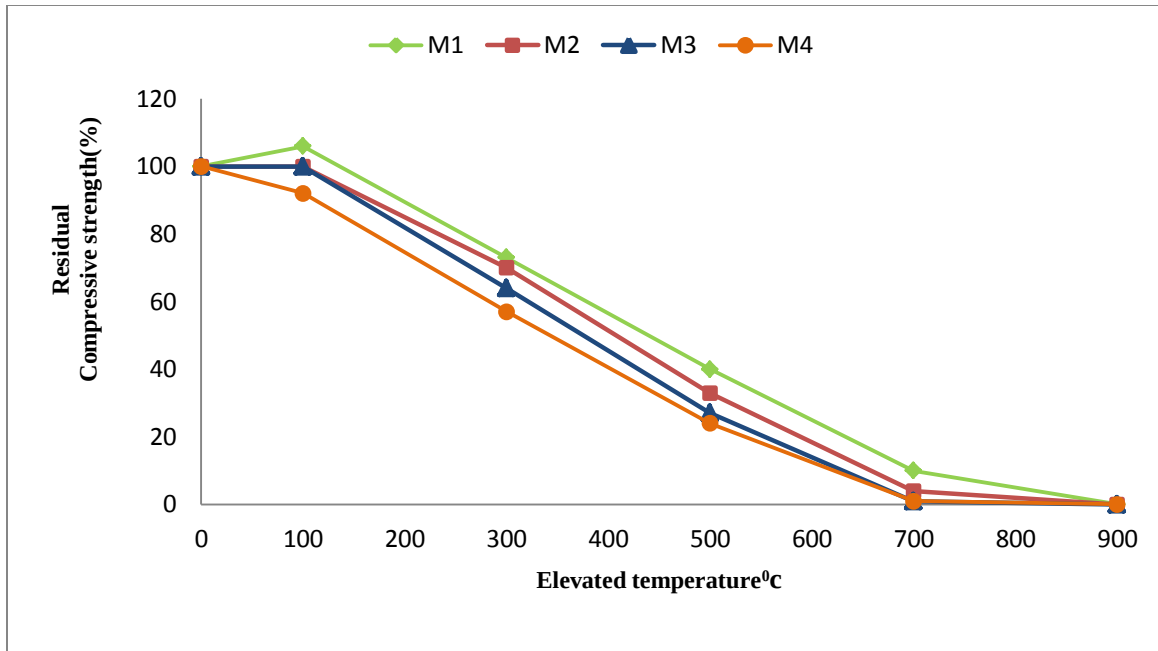


Fig.4.31 Retained compressive strength of 3-day age concrete @ 90mins exposure time

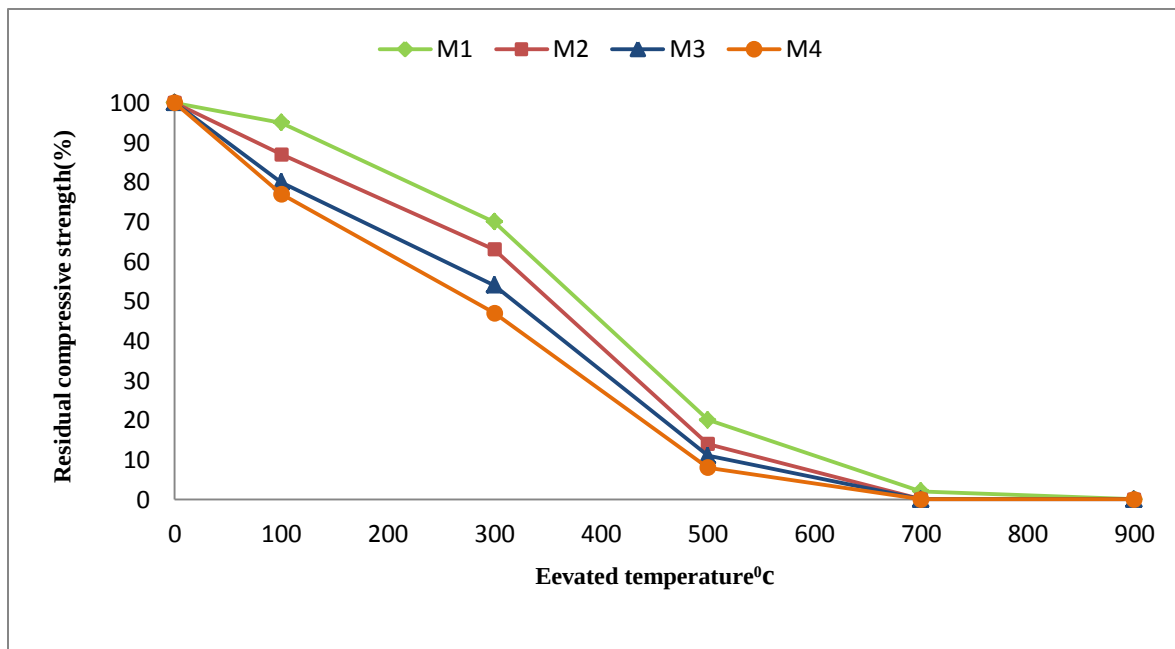


Fig.4.32 Retained compressive strength of 3-day age concrete @ 120mins exposure time

(iii)- (7 day age concrete)

The residual compressive strength values of 7 day age concrete test specimens after exposure to high temperature are presented in table 3.119, 3.124, 3.129 and 3.134 of chapter three. The retained strengths represent different aggregates concretes labeled M_1 , M_2 , M_3 and M_4 as earlier stated. The retained compressive strength values are represented on figures 4.39 to 4.42. As above, the retained compressive strength values significantly increased at exposure temperature of 100°C . However, the results slightly dropped to 85% below the original strength at 300°C . A similar pattern as 1 day age concrete was displayed, that heating to 100°C resulted in a noticeable increase in strength amounting to 18% and dropped to 85% of its original strength when the temperature was further raised to 300°C . The result of seven day age concrete showed an increase of 10% upon that of a 3 day age concrete. This shows that, as concrete age increases retained strength increases. The effects of exposure time can generally be observed across the whole figures that retained compressive strength decreases with increase in exposure time. For example, the retained compressive strength value at 60minutes (figure 4.40) was 85% at 300°C and became 80% at the same 300°C exposure temperature when the exposure time was further raised to 90minutes in fig. 4.41 respectively. It is evident that, raising the temperature resulted in further decrease in strength. For example, the retained compressive strength values recorded in fig. 4. 40 were 85% at 300°C and dropped to 20% when the temperature was further raised to 700°C . In all cases, M_1 displays high fire performance compare to other M_s and 28 day age concrete also has high fire resistance compare to rest ages.

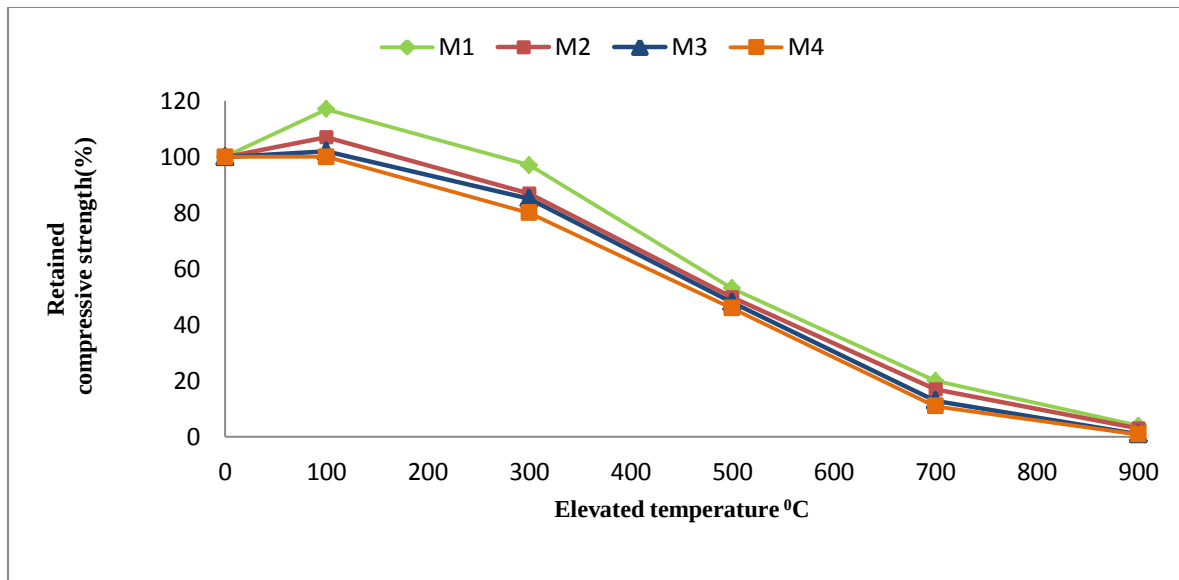


Fig.4.33 Retained compressive strength of 7-days old concrete @ 30mins exposure time.

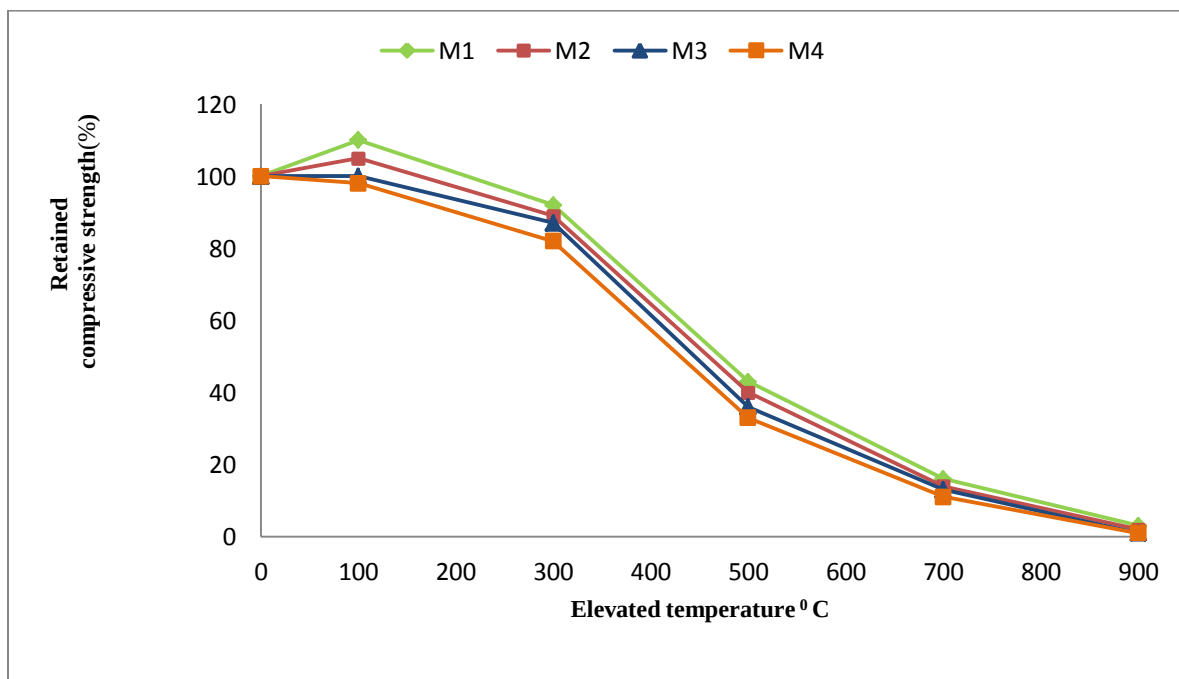


Fig.4.34 Retained compressive strength of 7-days old concrete @ 60mins exposure time.

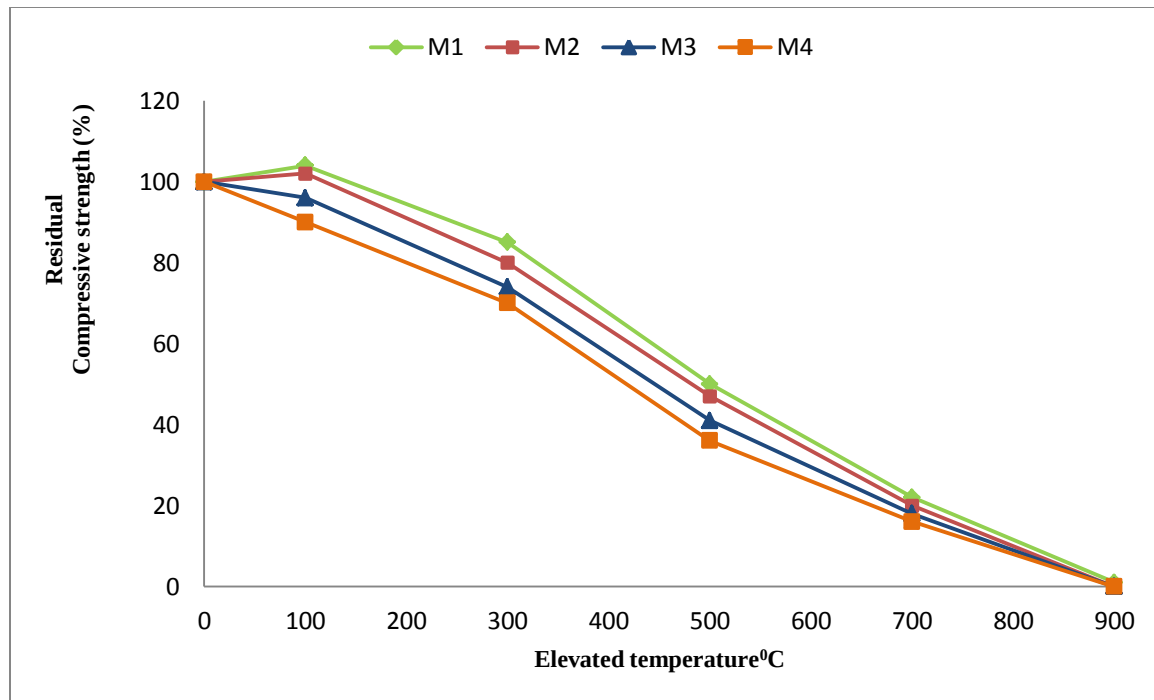


Fig.4.35 Retained compressive strength of 7-days old concrete @ 90mins exposure time.

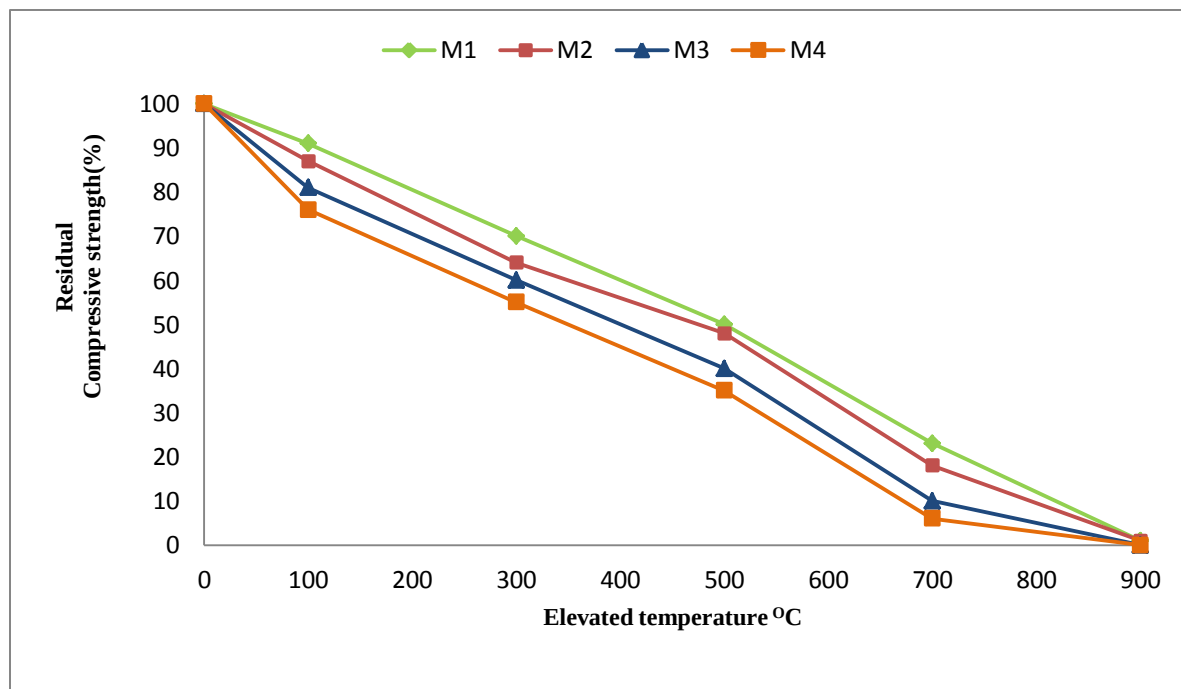


Fig.4.36 Retained compressive strength of 7-days old concrete @ 120mins exposure time.

IV- (14 and 28 day age concrete)

The residual compressive strength values of 14 and 28 day age concrete test specimens after exposure to high temperature are presented in table 3.120, 3.125, 3.130 & 3.135 and 3.121, 3.126, 3.131 & 3.136 of chapter three respectively. The retained strengths represent different aggregates concretes labeled M_1 , M_2 , M_3 and M_4 as stated earlier. The retained compressive strength values are represented on figures 4.43, 4.44, 4.45 & 4.46 and 4.47, 4.48, 4.49 and 4.50. An interesting observation is that, the specimens at 14 and 28 day age have not benefited much to the increase in the retained compressive strength values at exposure temperature of 100°C as early aged concrete, however, the compressive strength values at high temperature appeared to be high than that of the early aged concrete(1, 3 and 7 days age). For example, the average residual compressive strengths for 14 and 28 day aged concrete at 100°C for 30min.(fig. 4.43 and 4.47) exposure time are amounting to 107% and 105% respectively. However, when the exposure temperature was raised to 300°C and maintained for that 30minutes, the results was found to be 90% and 93% for 14 and 28 days age respectively. A similar trend was maintained that, M_1 appeared to display high fire performance. It is also observed that, none of the specimens crumbled to zero at all exposure temperatures and exposure times. This shows that, as concrete age increases the retained compressive strength increases as well. The effects of exposure time can generally be observed across the whole figures that retained compressive strength decreases with increase in exposure time. For example, the retained compressive strength value at 30minutes (figure 4.43 and 4.47) was 90% and 93% at 300°C and became 67% & 80% at the same 300°C when the exposure time was raised to 120minutes (fig. 4.46 & 4.50) respectively.

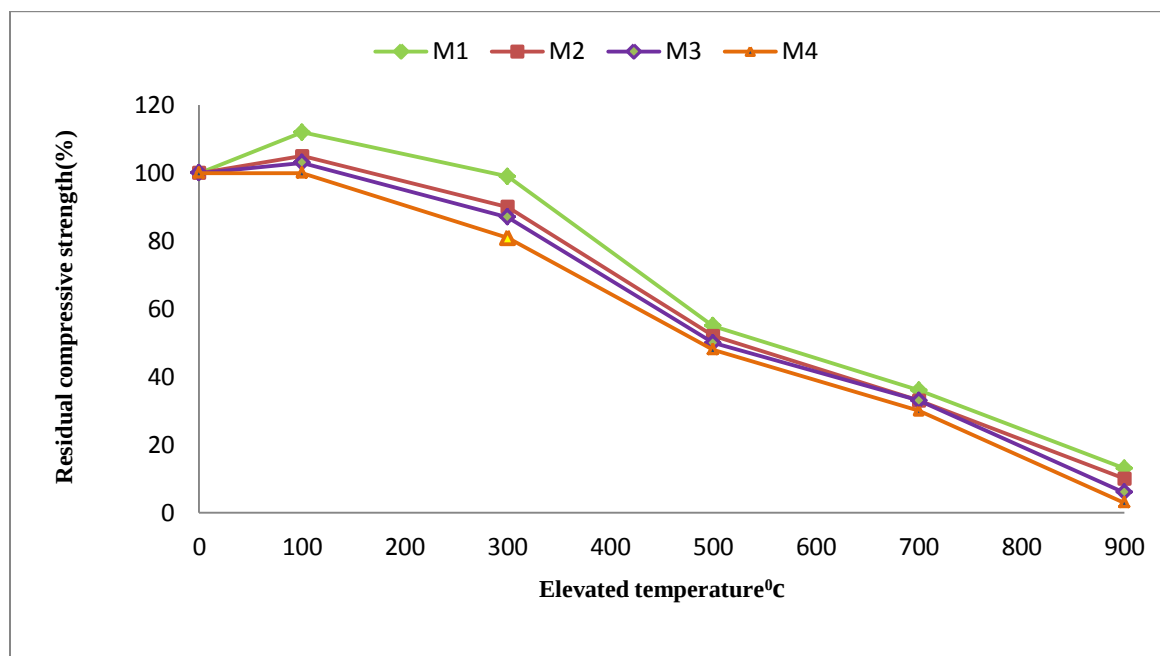


Fig.4.37 Retained compressive strength of 14-days old concrete @ 30mins exposure time.

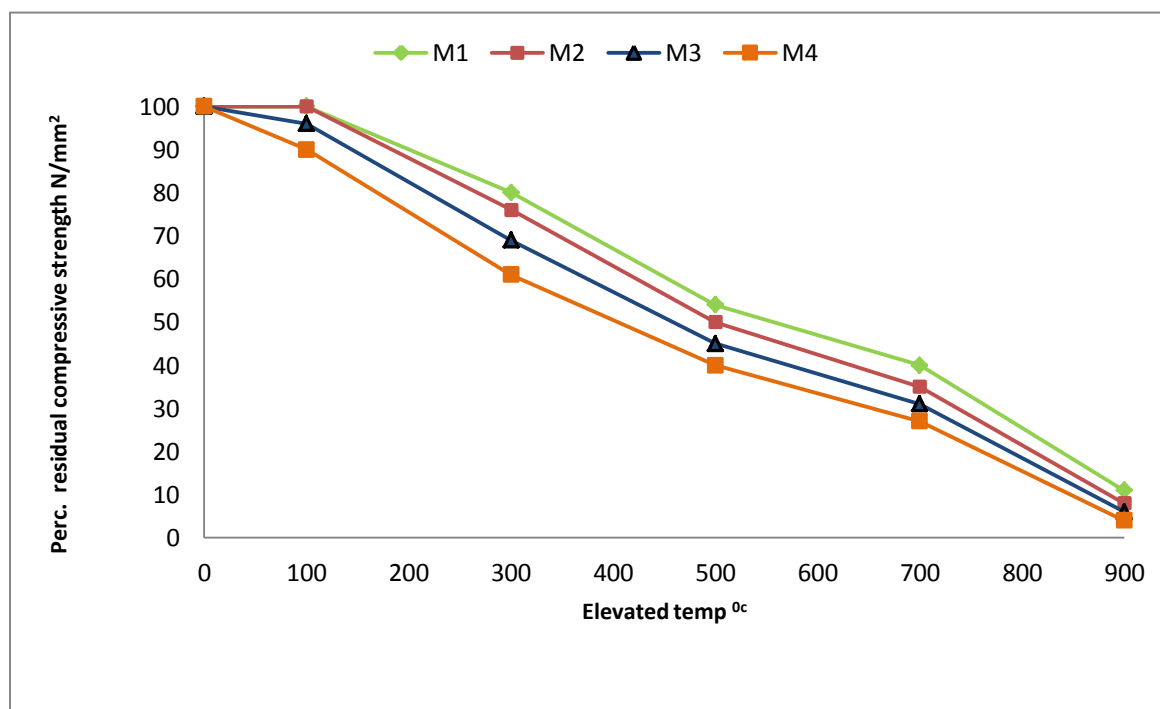


Fig.4.38 Retained compressive strength of 14-days old concrete @ 60mins exposure time.

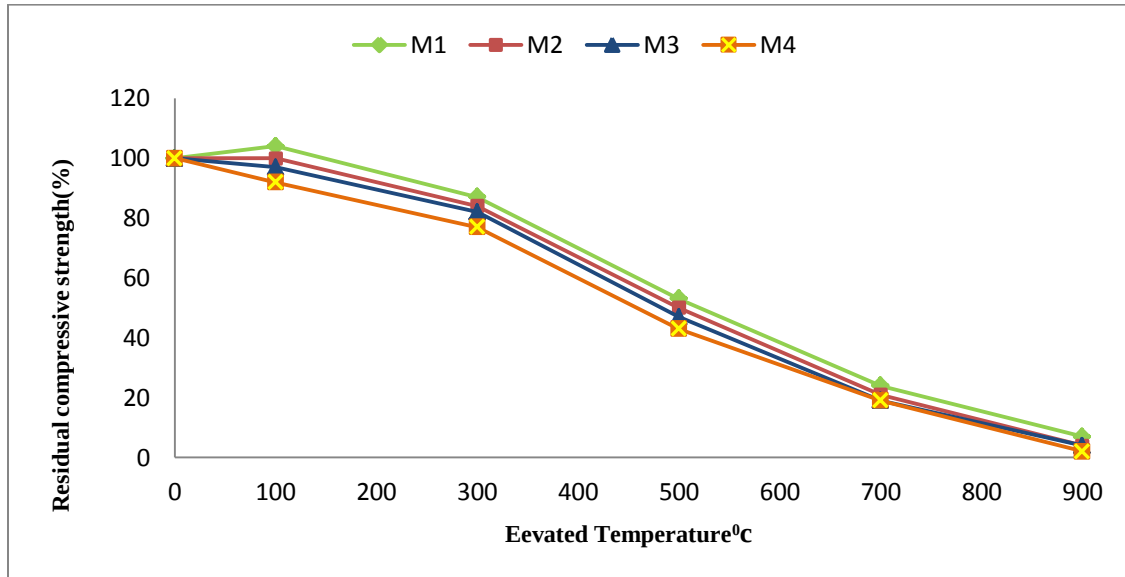


Fig.4.39 Retained compressive strength of 14-days old concrete @ 90mins exposure time.

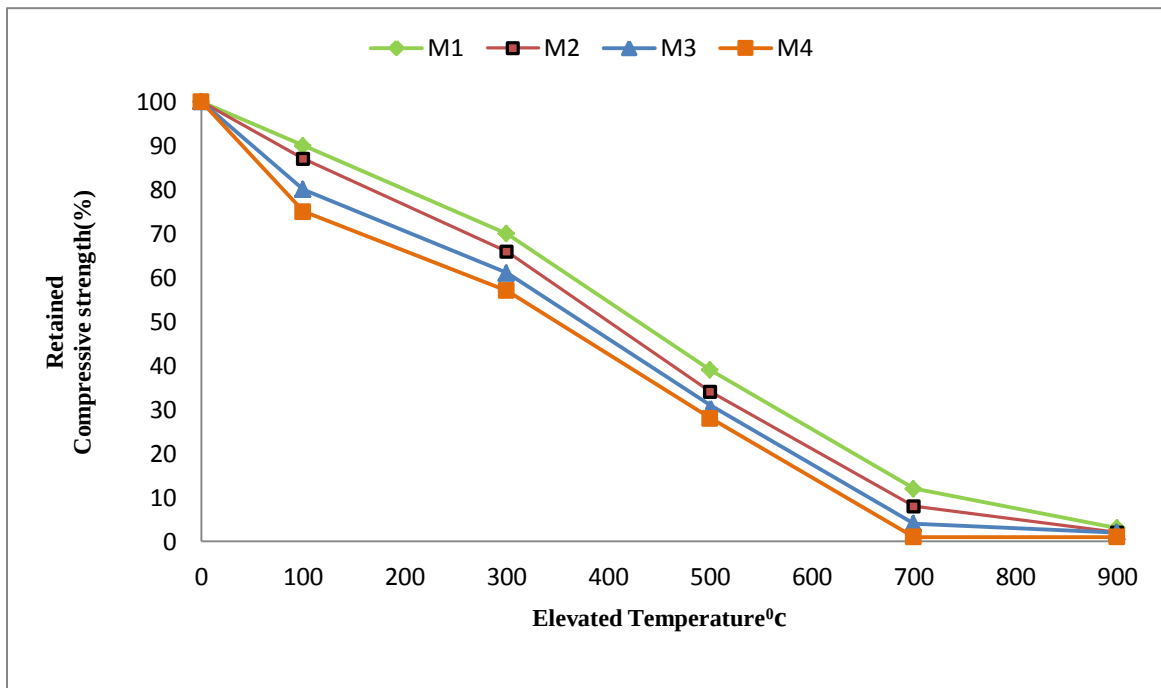


Fig.4.40 Retained compressive strength of 14-days old concrete @ 120mins exposure time.

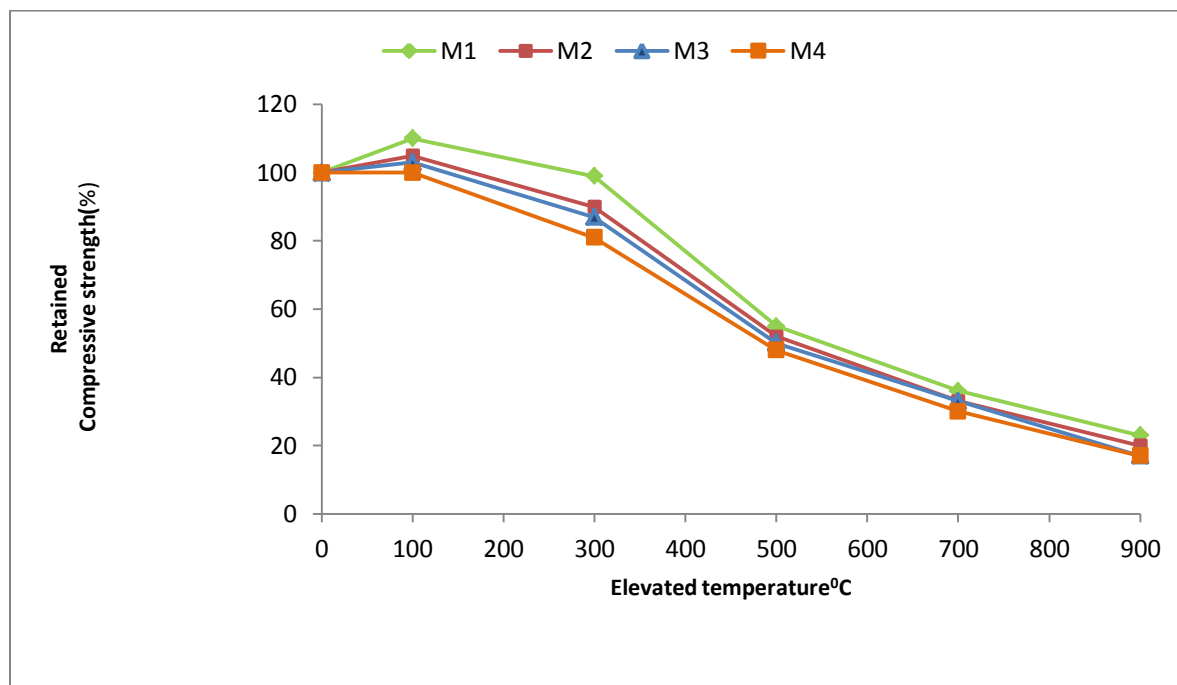


Fig.4.41 Retained compressive strength of 28-days old concrete @ 30mins exposure time.

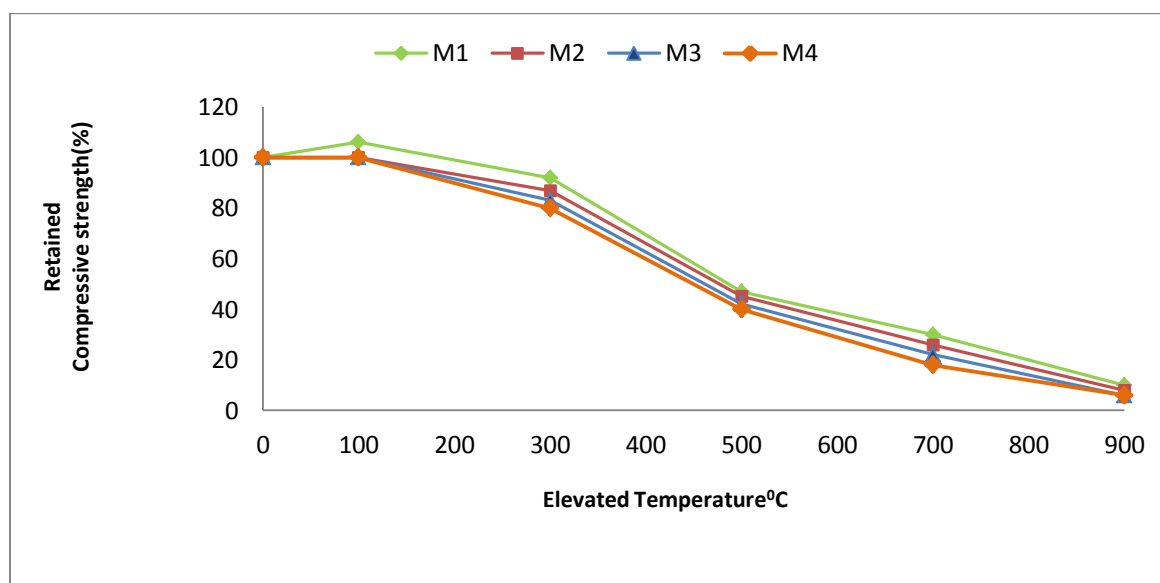


Fig.4.42 Retained compressive strength of 28-days old concrete @ 60mins exposure time

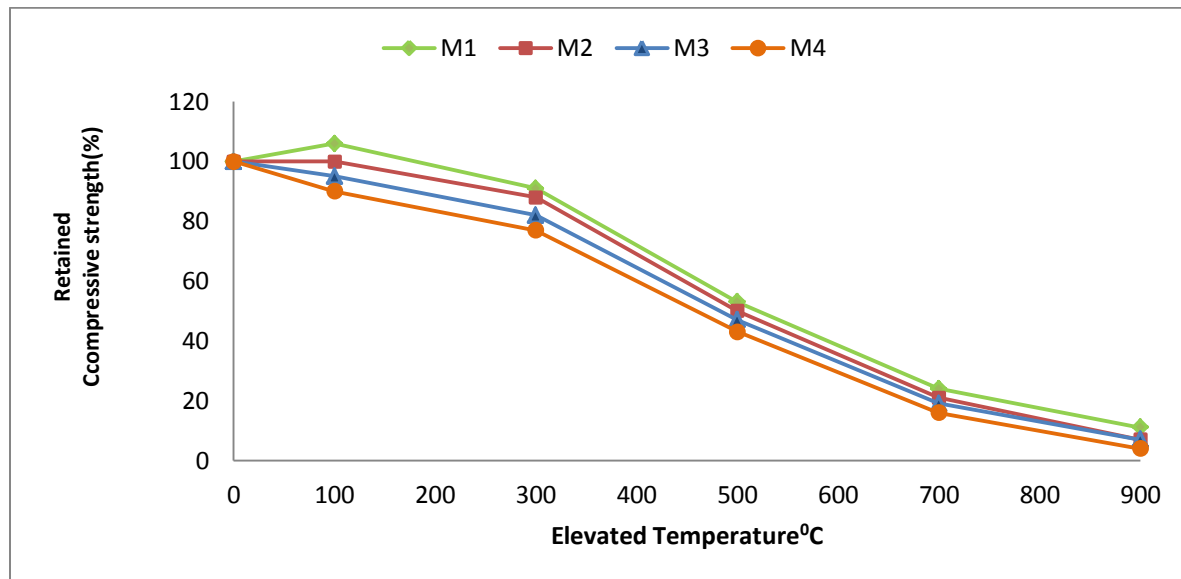


Fig.4.43 Retained compressive strength of 28- days' old concrete @ 90mins exposure time.

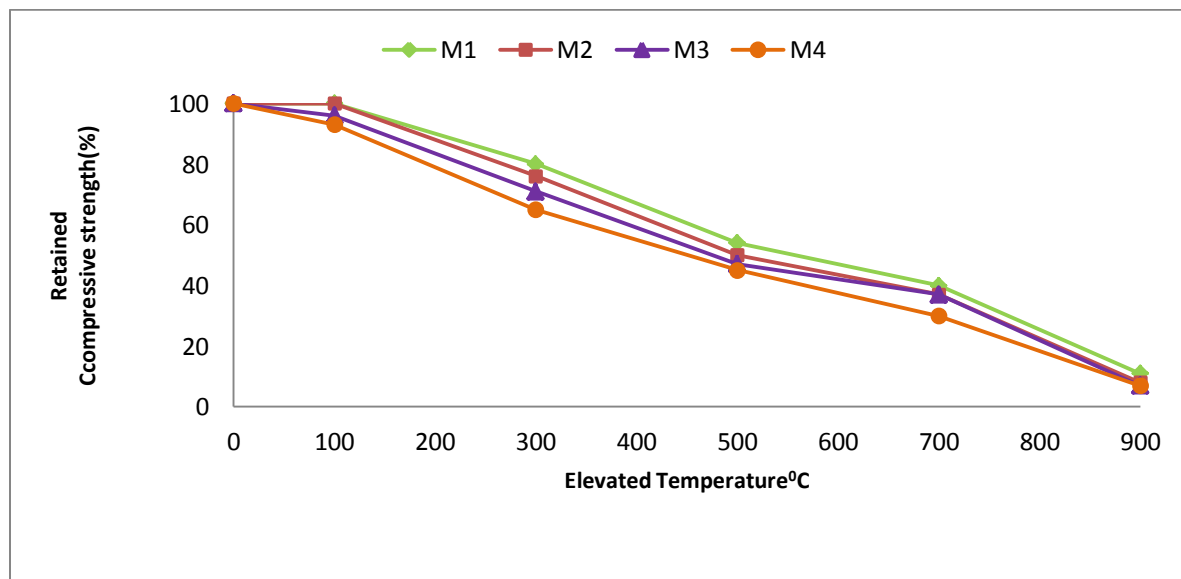


Fig.4.44 Retained compressive strength of 28- days old concrete @ 120mins exposure time.

4.9: Zero Strength Exposure Time and Temperature

Tables 4.10 to 4.14 show the temperatures and exposure times at which the Concrete Strength is zero. As can be seen observed in table 4.10, as the exposure time increases, the temperature at which the Compressive strength is zero also decreases. Same procedure for all the zero strength table show in table 4.10 to 4.14

Table 4.10: Zero Strength Exposure Time and Temperature for 1-Day Age Concrete

S/No	Exposure Time (mm)	Exposure Temperature (°C)
1	30	900
2	60	700
3	90	650
4	120	500

Table 4.11: Zero Strength Exposure Time and Temperature for 3-Days age Concrete

S/No	Exposure Time (mm)	Exposure Temperature (°C)
1	30	850
2	60	850
3	90	850
4	120	700

Table 4.12: Zero Strength Exposure Time and Temperature for 7-Days age Concrete

S/No	Exposure Time (mm)	Exposure Temperature (⁰ C)
1	30	900
2	60	900
3	90	900
4	120	900

Table 4.13: Zero Strength Exposure Time and Temperature for 14-Days age Concrete

S/No	Exposure Time (mm)	Exposure Temperature (⁰ C)
1	30	≥900
2	60	≥900
3	90	≥900
4	120	≥900

Table 4.14: Zero Strength Exposure Time and Temperature for 28-Days age Concrete

S/No	Exposure Time (mm)	Exposure Temperature ($^{\circ}\text{C}$)
1	30	>900
2	60	>900
3	90	>900
4	120	>900

4.10 Residual Deflection

When a beam is loaded, it deflects from its original position. The amount, by which a beam deflects, depends upon its cross- section, stiffness and the bending stress. The beam specimens in this work were subjected to high temperatures which were found to have negatively affected the stiffness of the reinforced concrete beam specimens.

4.10.1 Load-Deflection Curve

The values of residual deflection of 28 day age reinforced concrete specimens after exposure to high temperature are presented in tables 3.137 to 3.142 of chapter three.

The effect of elevated temperature on reinforced concrete beams (deflection) in respect of different concrete cover is displayed in Load- deflection curves as shown in Figs 4.51 to 4.56 and the summary of the analysis for both 60minutes and 120minutes is presented in tables 4.15 to 4.20 below.

(i) Sixty minutes Exposure (60min)

In this, the concrete specimens were exposed to different elevated temperatures of 100⁰c, 300⁰c and 600⁰c and were maintained at these temperatures for the period of 60 minutes. Three different concrete covers of 20mm, 25mm and 30mm were used for the assessment of residual deflection in this experiment. The results show that, 20mm cover gives the highest residual deflection and 30mm cover gives the least at all temperature as displayed on figure 4.51 to 5.53 respectively. This is an indication that, concrete cover plays significant role in service durability of reinforced concrete in fire.

The recorded maximum deflection at 100⁰c (table 4.15below) for the respective concrete covers maintained for 60minutes are 14.7mm, 20mm and 23mm respectively. When the temperature was further raised to 300⁰c at the same exposure time of 60minutes (See table 4.16 below) the deflection increases to 20mm, 26mm and 30mm respectively. The recorded deflection at 600⁰C maintained for 60minutes for the respective concrete covers are 31.0mm, 27.0mm and 24mm respectively. (Table 4.17 below)

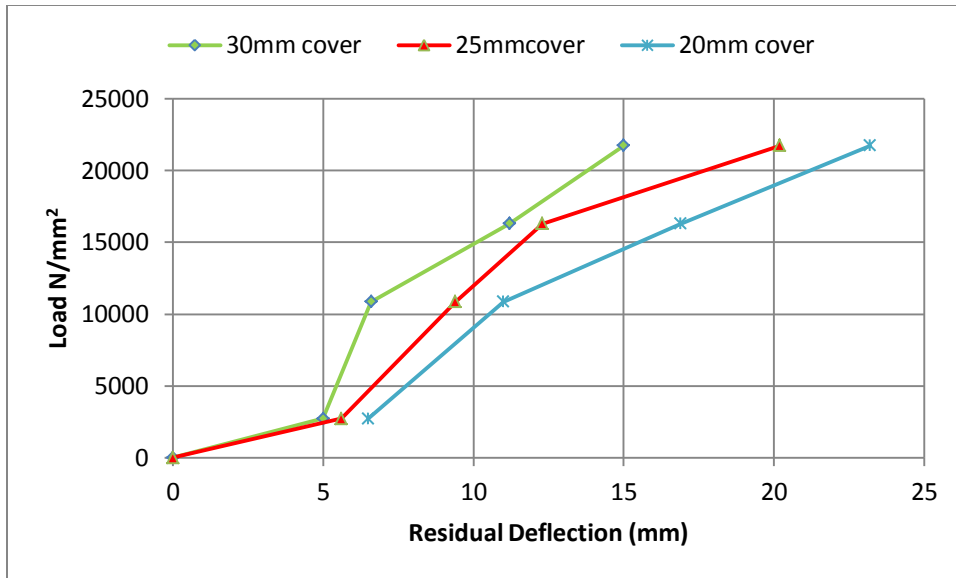


Fig 4.45- Variation of residual deflection with temperature at 100°C for 60min

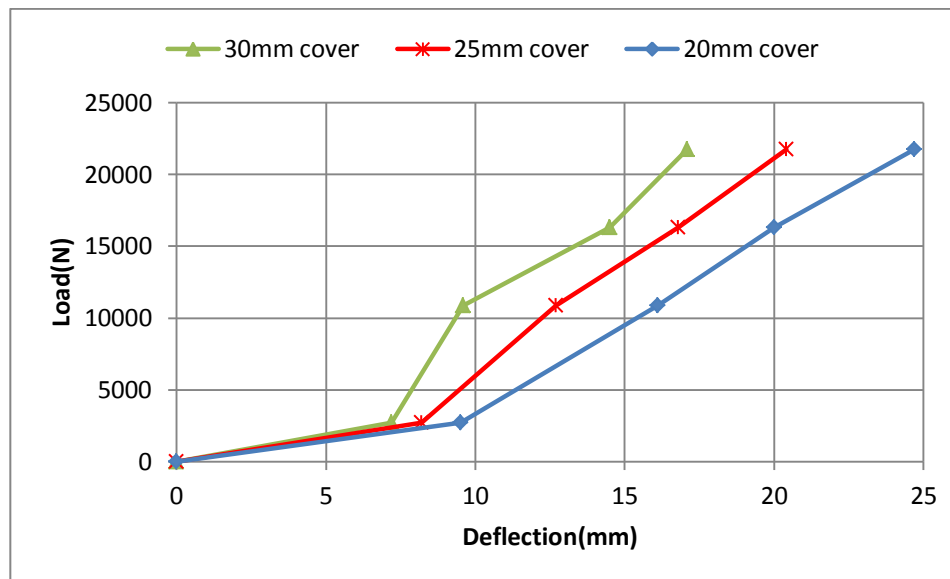


Fig 4.46- Residual deflection with temperature at 300°C for 60min

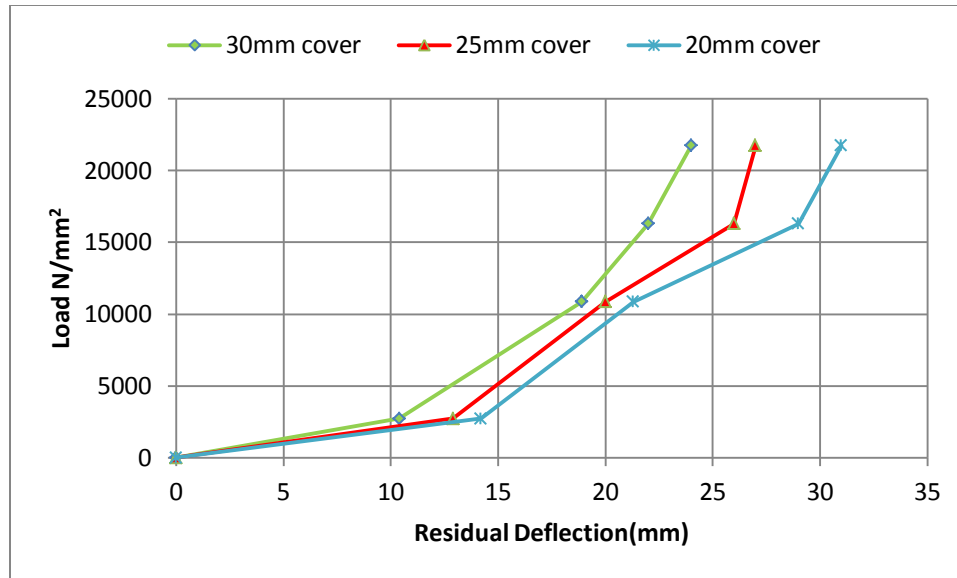


Fig 4.47- Residual deflection with temperature at 600°C for 60min

Table 4.18 deflection of different concrete cover @ 100°C, 60mins duration

Concrete cover → Deflection ↓ Load(N)	20mm residual def(.mm)	25mm residual def.(mm)	30mm residual def.(mm)
2716.2	6.6	5.6	5.1
10864.8	11.2	9.5	6.8
16297.2	16.9	12.3	11.2
21729.6	23.0	20.0	14.3

Table 4.16 deflection of different concrete cover @ 300⁰c, 60mins duration

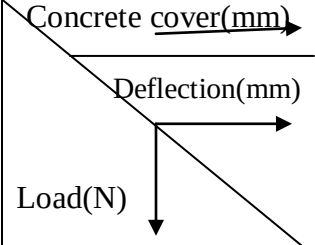
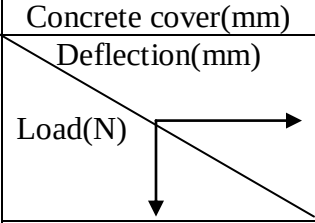
	20mm	25mm	30mm
	residual deflection (mm)	residual deflection (mm)	residual deflection (mm)
2716.2	9.5	8.2	7.2
10864.8	20	13.6	9.6
16297.2	26.2	19.5	14.2
21729.6	30.0	26.0	20

Table 4.17 Residual deflection of different concrete cover @ 600⁰c, 60mins duration

	20mm	25mm	30mm
	residual def.mm	residual def.mm	residual def.mm
2716.2	14.2	12.9	10.4
10864.8	21.3	20.0	18.9
16297.2	29.0	26	22
21729.6	31.0	27.0	24

(ii) One Hundred and Twenty Minutes Exposure Time (120minutes)

The values of residual deflection of 28 day age reinforced concrete specimens after exposure to high temperature are presented in tables 3.137 to 3.142 of chapter three.

The effect of elevated temperature on reinforced concrete beams (deflection) in respect of different concrete cover is displayed in Load- deflection curves as shown in Figs 4.54 to 4.56 below. The combined effects are also displayed in figure 4.57 to 4.59 below. The summary of the analysis for both 60minutes and 120minutes is presented in tables 4.15 to 4.20 below.

At this period of time (120minutes), the concrete specimens were exposed to 100⁰c, 300⁰c and 600⁰c and were maintained for the period of 120minutes. Different concrete cover of 20mm, 25mm and 30mm were also used for the assessment of residual deflection. The results show that, 20mm cover gives the highest residual deflection and 30mm cover gives the least at all temperatures and exposure period. The maximum deflection at 100⁰c for the respective concrete covers of 20mm, 25mm and 30mm are 29.6mm, 22mm and 20mm (table 4.18) respectively. When the temperature was further raised to 300⁰c at the same exposure time of 120minutes, (table 4.19) the deflection increases to 35mm, 32mm and 28mm respectively. The recorded deflection at 600⁰C maintained for 120minutes for the respective concrete covers is amounting to 40mm, 34mm and 31mm, respectively. These results agree with the work of Man sour et, al (2009), Hong et al (2002). and Rahul et, al (2012)

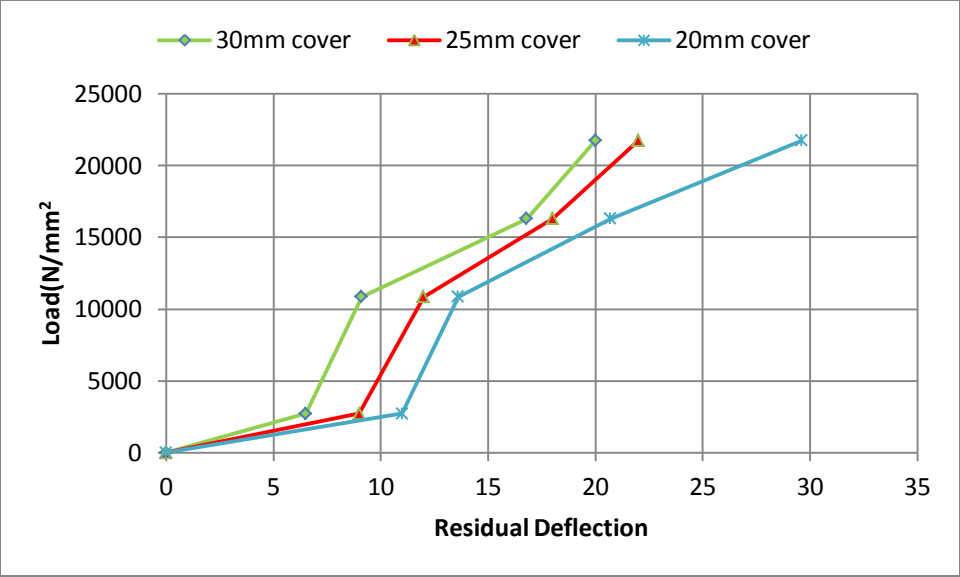


Fig 4.48- Residual deflection with temperature at 100°c for 120min

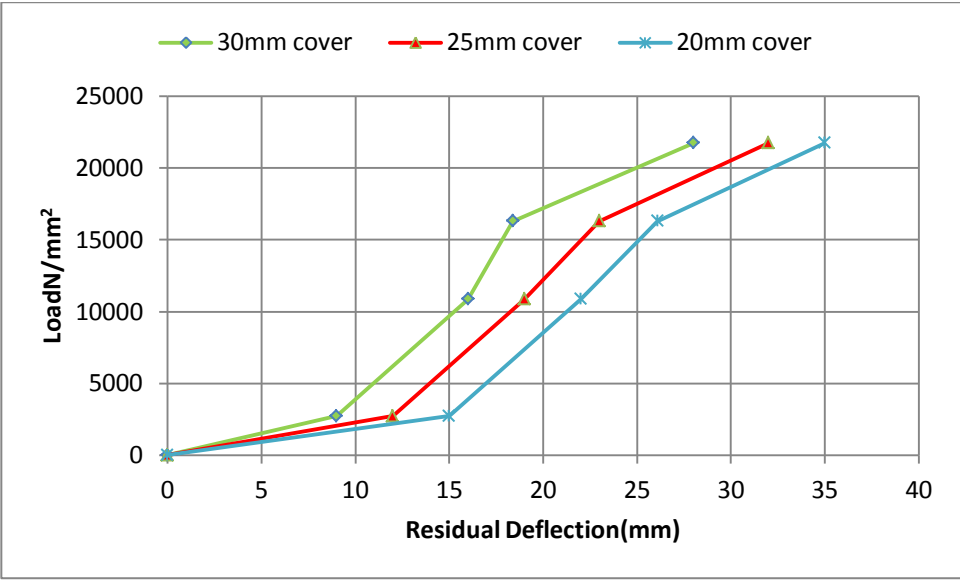


Fig 4.49- Residual deflection with temperature at 300°c for 120min

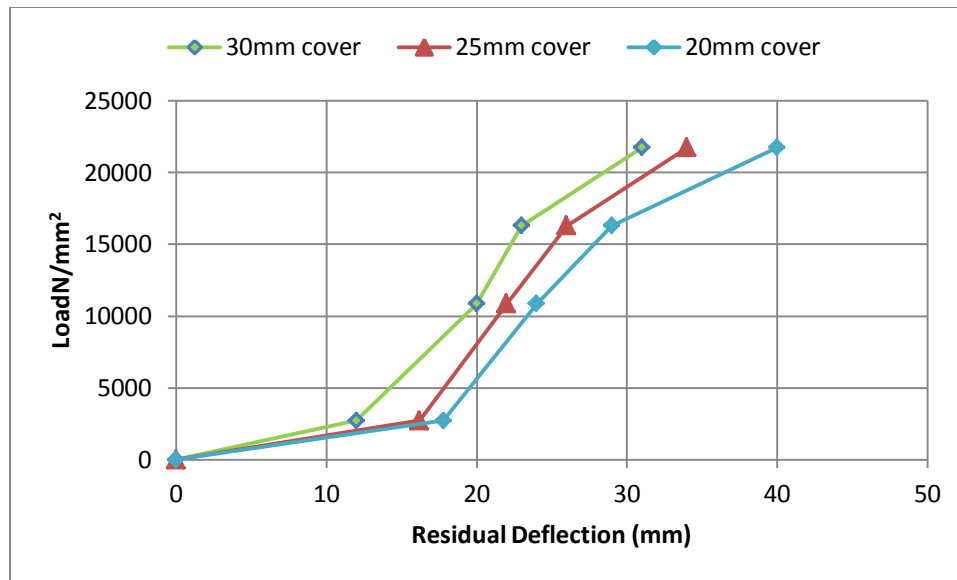


Fig 4.50- Variation of residual deflection with temperature at 600°c for 120min

Table 4.18 deflection of different concrete cover @ 100°c, 120mins duration

Concrete Cover(mm)	20mm	25mm	30mm
Deflection(mm)	residual def.mm	residual def.mm	residual def.mm
Load(N)			
2716.2	11.0	9.0	6.5
10864.8	13.6	12.0	9.1
16297.2	20.7	18.0	1.8
21729.6	29.6	22.0	20

Table 4.19 deflection of different concrete cover @ 300⁰c, 120mins duration

Concrete Cover(mm) →	20mm	25mm	30mm
Deflection(mm) ↙ Load(N) ↓	residual def.mm	residual def.mm	residual def.mm
2716.2	15.0	12.0	9.0
10864.8	22.0	19.0	16.0
16297.2	26.0	23.0	18.4
21729.6	35.0	32.0	28.0

Table 4.20 deflection of different concrete cover @ 600⁰c, 120mins duration

Concrete Cover(mm)	20mm	25mm	30mm
Deflection(mm) ↙ Load(N) ↓	residual def.mm	residual def.mm	residual def.mm
2716.2	17.8	16.2	12.0
10864.8	24.0	22.0	20.0
16297.2	29.0	26.0	23.0
21729.6	40.0	34.0	31.0

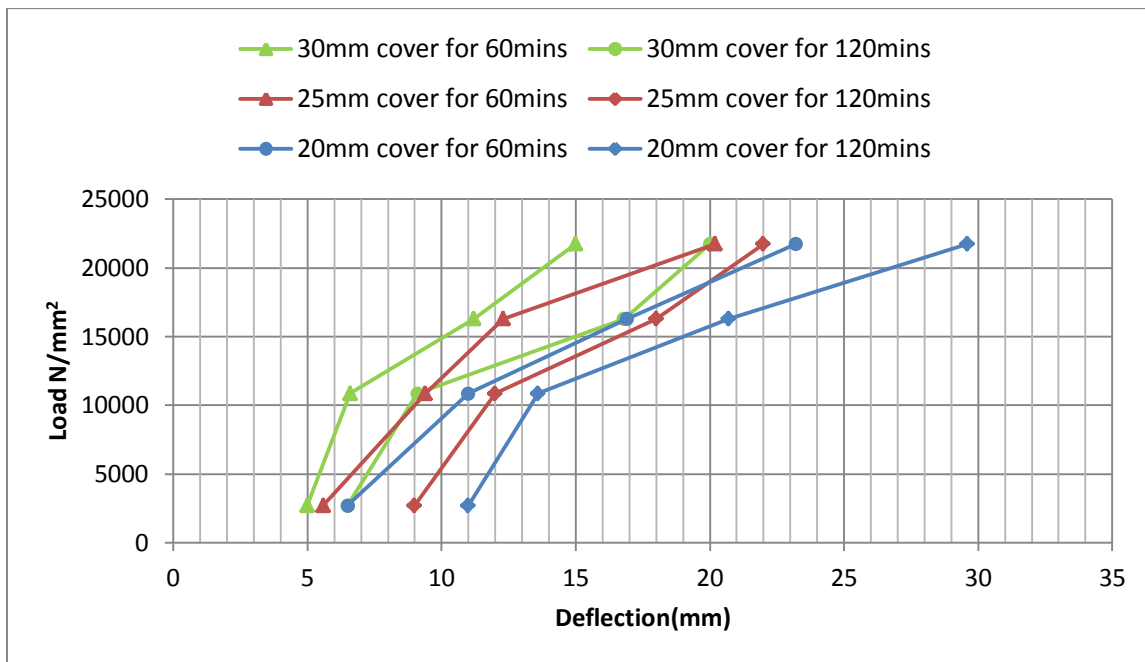


Fig. 4.51 Combined residual deflection of concrete specimen exposed to 100°C

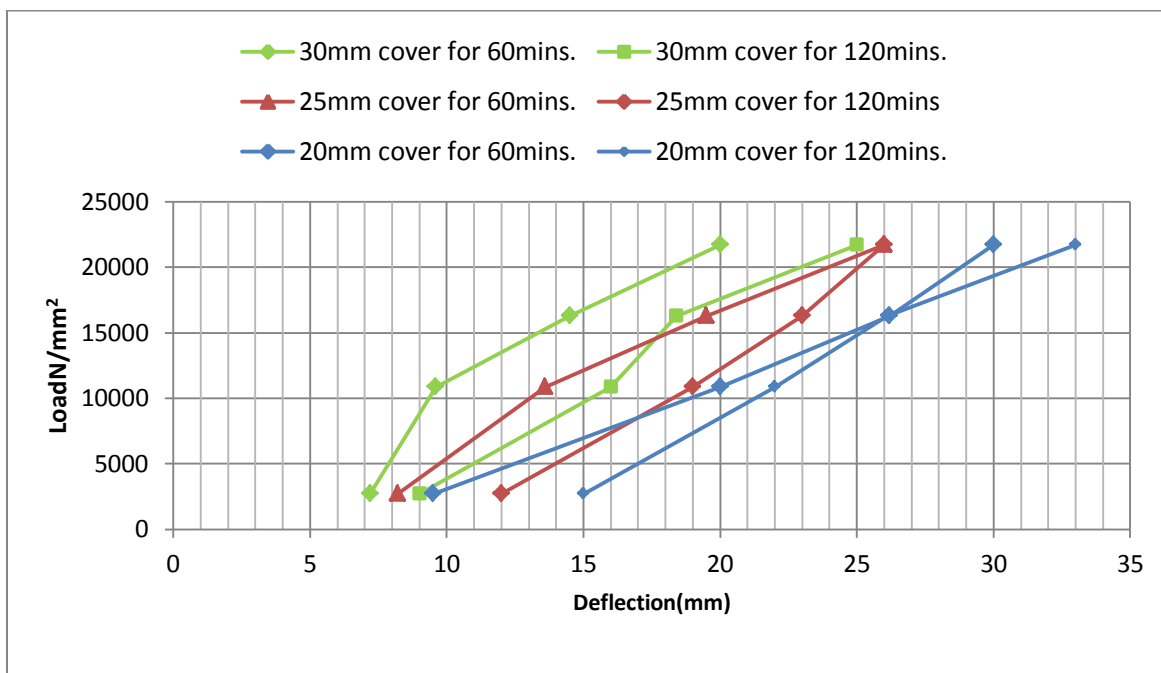


Fig. 4. 52 Combined residual deflection of concrete specimen exposed to 300°C

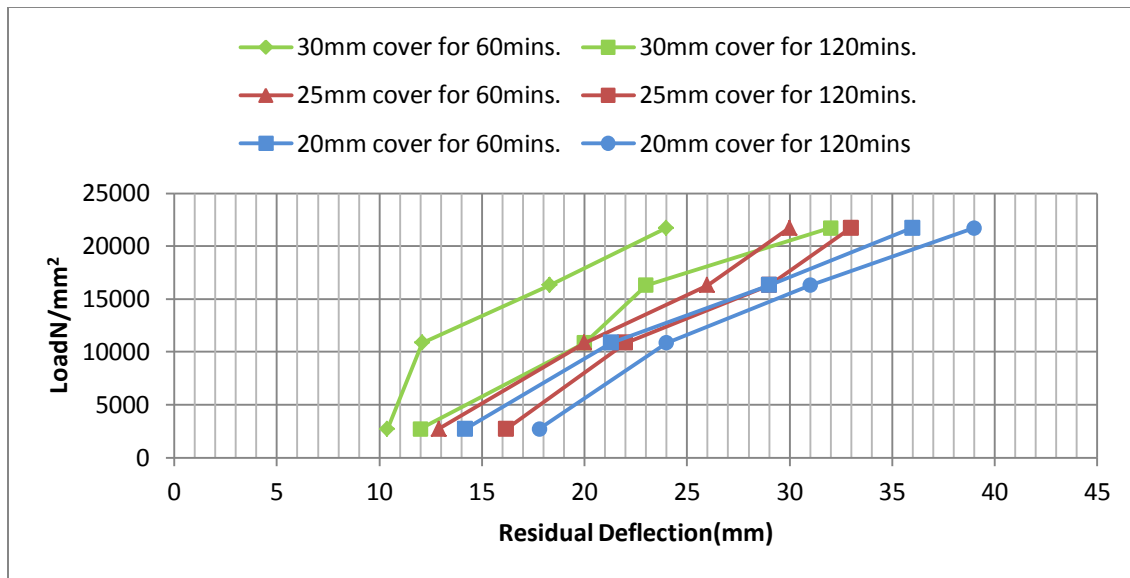


Fig. 4.53 Combined residual deflection of concrete specimen exposed to 600°C

Figure 4.56 to 5.59 show the combined plot of the variation of residual deflection with exposure periods for 60 and 120min respectively. The result comparatively shows significant variation of residual deflection with varying exposure time across different cover to reinforcement with the same exposure temperature. The results show that, 20mm cover always yields larger deflection and 30mm produces less in all cases as provided by the BS8110, 1997 code of practice.

4.11 Elastic Modulus of Reinforced Concrete

Elastic modulus (ASTM C469-a (2000)) which is a measure of toughness of material is a very significant parameter to carry research on. The bending stiffness (EI_x) of a beam is the product of two variables: (1) second moment of area I_x , which is reflecting the cross-sectional resistance to loading; and (2) the modulus of elasticity (E), reflecting the material resistance to loading $\dot{I}$. Kalkan, (2010). In fire, both variables are subject to change during the course of fire loading. The variation in the modulus of elasticity with the increasing fire load is caused by the inelastic stress-strain behavior of concrete beyond the elastic limits, while the variation in the moment of inertia is associated with the cracking normally at critical temperature due to the flexural stress.

The cracked zones in a concrete beam are ineffective in resisting stresses originating from fire loads. Therefore, decrease in elastic modulus results in decreasing resistance of a concrete beam to loading, leading to greater deformations in the beam. Variation of residual elastic modulus with different elevated temperature and exposure time is shown on Fig. 4.65 and 4.66 below respectively.

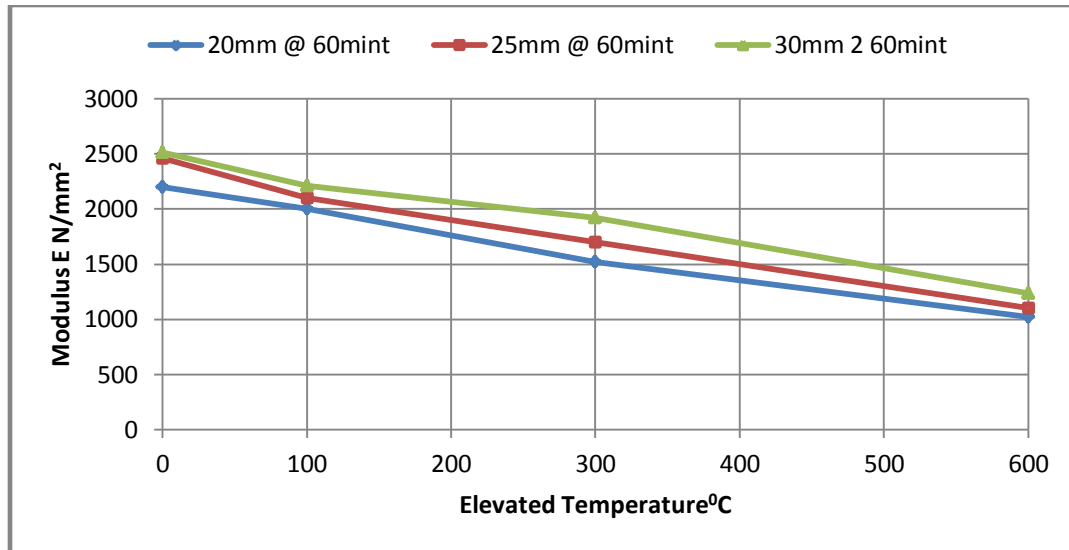


Fig.4.54 Modulus of Elasticity at 60minutes

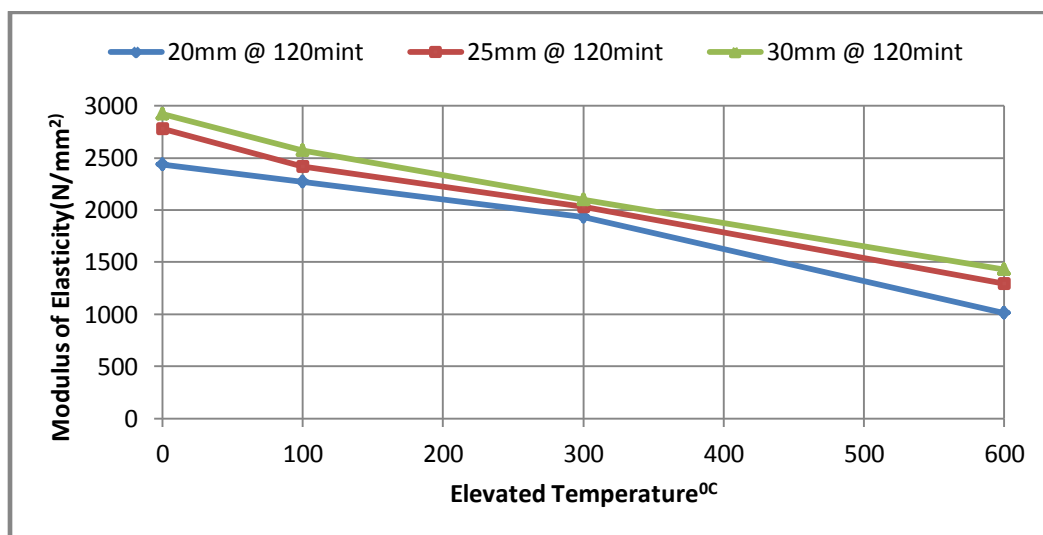


Fig.4.55 Modulus of Elasticity at 120minutes

The specimens were exposed to different elevated temperatures and maintained for 60 and 120minutes duration. In the above figures, it is clear that, elastic modulus decreases with increasing temperature and exposure time. For example, at 60minutes exposure time, the elastic modulus for the respective concrete covers at 100⁰c is (2268, 2420 and 2568N/mm²). When the temperature was raised to 300⁰c, the respective elastic modulus at 60minutes exposure is (1932, 2031 and 2100N/mm² corresponding to 20mm, 25mm and 30mm concrete cover respectively. Further heating of specimens to 600⁰c, the recorded elastic modulus for 20mm, 25mm and 30mm concrete cover is (1011, 1293 and 1432N/mm²). However, when the exposure period was raised to 120minutes, the respective elastic modulus recorded in respect of 30mm, 25mm and 20mm concrete cover at 100⁰C, 300⁰C and 600⁰C are (2002, 2100 and 2212N/mm²), (1520, 1700 and 1920N/mm²) and (1020, 1100 and 1232N/mm²) respectively. This result is in line with the finding of (Ali Nadjai, 2005) and (Rahul.P. 2012), from the results, it is clear that elastic modulus decreases with increase in temperature and exposure time. From the plots, it shows that, the bigger the concrete cover, the high the performance of concrete in fire. Fig.4. 65 and 4.66 and table 4.21 to 4.22 show the results of elastic modulus in respect of temperature and exposure times below.

Table 4.21 Summary of elastic modulus analysis at 60min. exposure

Concrete Cover	Temperature	Exposure	EN/mm ²
20mm	Ambient Temperature (0 ⁰ C)		2432
25mm			2780
30mm			2920
20mm	100 ⁰ C	60min	2268
25mm			2420
30mm			2568
20mm	300 ⁰ C		1932
25mm			2031
30mm			2100
20mm	600 ⁰ C		1011
25mm			1292
30mm			1432

Table 4.22 Summary of elastic modulus analysis at 120min. exposure

Concrete Cover (mm)	Temperature ⁰ C	Exposure (min)	E N/mm ²
20mm	Ambient Temperature ⁰ C		2200
25mm			2460
30mm			2511
20mm	100 ⁰ C	120min	2002
25mm			2100
30mm			2212
20mm	300 ⁰ C		1520
25mm			1710
30mm			1900
20mm	600 ⁰ C		1020
25mm			1100
30mm			1232

4.12 Bending Strength of Reinforced Concrete Beams in Fire

The results of residual bending strength at 60 and 120minutes duration under temperature are presented on table 3.145 to 3.146 of chapter three and now represented in Fig. 4.67 and 4.68 below.

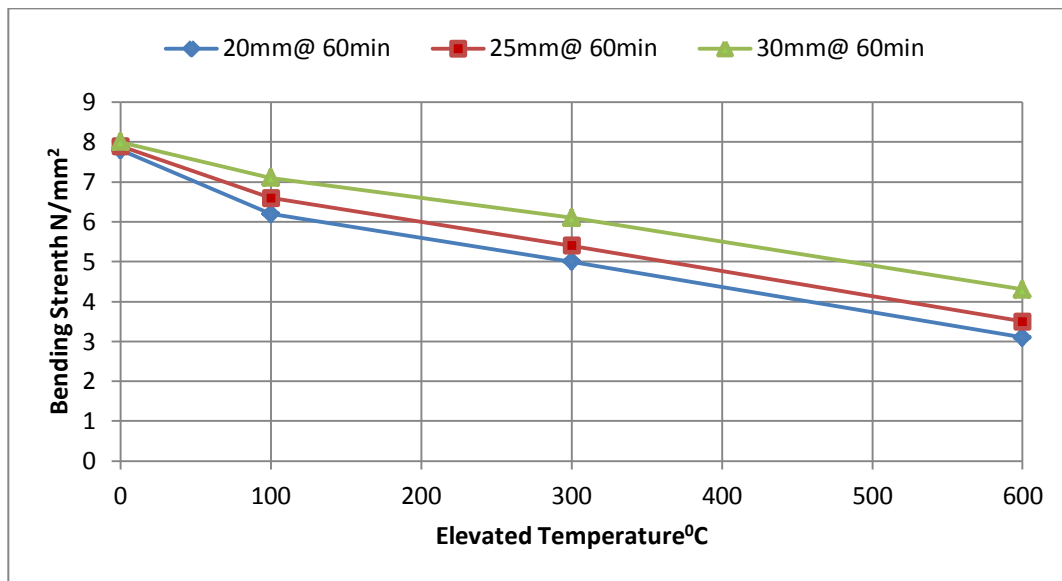


Fig.4.56 Residual Bending Strength of concrete prism at 60mins

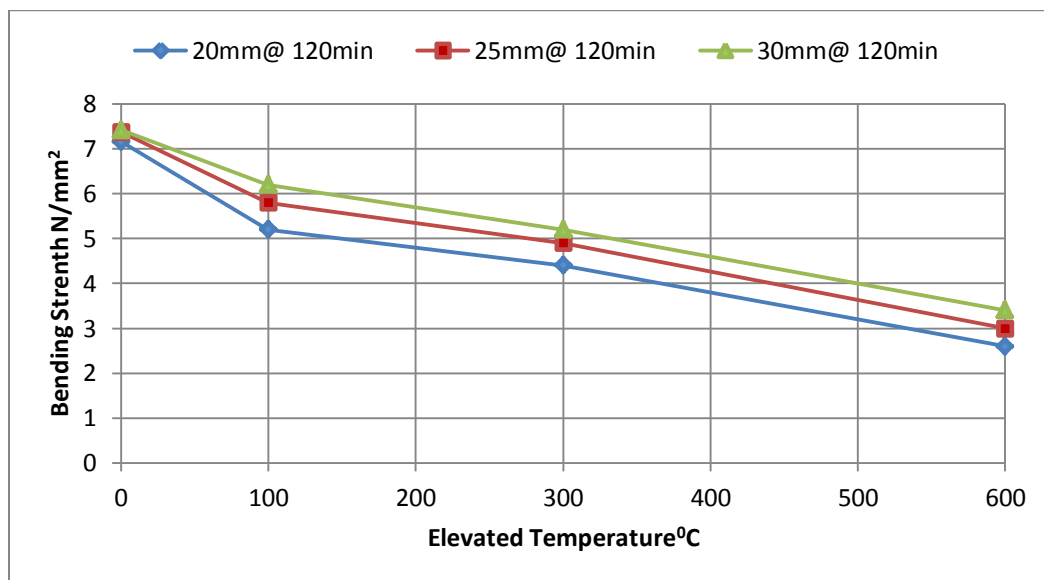


Fig.4.57 Residual Bending Strength of concrete prism at 120mins

The plots show the variation of residual bending strength with elevated temperature for 20, 25 and 30mm concrete cover. The specimens were subjected to three (3) different elevated temperatures and were maintained at those temperatures for 60 and 120minutes respectively. From the said figures, it is very clear that, as the temperature rises, bending strength declines accordingly, for instance, in figure 4.67; the respective bending strength at 100⁰c is 6.2, 6.6 and 7.1N/mm² for different concrete covers. When the specimens were further heated to 600⁰c, the following bending strengths were recorded 3.1, 3.5 and 4.3N/mm² in respect of 20mm, 25mm and 30mm concrete cover. When the exposure time was now raised to 120minutes the value of bending strength further decreases in accordance to size of concrete cover, for example in Fig. 4.68, the following values of bending strength (5.2, 5.8 and 6.2N/mm²) and 2.6, 3.0 and 3.4N/mm²; were recorded at 100⁰c and 300⁰c in respect of 20mm, 25mm and 30mm concrete covers. Similar values were obtained by Mansour et, al, (2007) and Ahmed et, al. (2004). It also agrees with the work of Rahul. (2012), Ali, (2005) and Kalkan, (2010)

On a general note, it is clear that bending capacity of concrete in fire drastically reduces as the temperature and exposure time increases. The results generally show the influence of concrete cover in respect of heat insulation. 30mm concrete cover shows higher performance in fire. This is followed by 25mm concrete cover. While the weakest is the 20mm concrete covers. Conclusively, the bigger the concrete cover, the higher the performance of concrete in fire.

Fig.4. 67 and 4.68 show the results of bending strength in percentage. As above, the results show that, percentage bending strength decreases with increase in temperature and exposure times.

Table 4.23 Analysis of Bending Strength at 60min. exposure

Concrete Cover (mm)	Temperature ⁰ C	Exposure (min)	β (N/mm ²)
20mm	Ambient Temperature ⁰ C		7.8
25mm			7.9
30mm			8.1
20mm	100 ⁰ C	60min	6.2
25mm			6.6
30mm			7.1
20mm	300 ⁰ C		5.1
25mm			5.4
30mm			6.1
20mm	600 ⁰ C		3.1
25mm			3.5
30mm			4.1

Table 4.24 Analysis of Bending Strength at 120min. exposure

Concrete Cover (mm)	Temperature ⁰ C	Exposure (min)	β (N/mm ²)
20mm	Ambient Temperature ⁰ C		7.7
25mm			7.9
30mm			8.3
20mm	100 ⁰ C	120min	5.2
25mm			5.8
30mm			6.1
20mm	300 ⁰ C		4.4
25mm			4.9
30mm			5.2
20mm	600 ⁰ C		2.6
25mm			3.0
30mm			3.4

CHAPTER FIVE:
CONCLUSION AND RECOMMENDATION

5.1 CONCLUSION.

From the analysis of Results presented in chapter four and the results in chapter three. Generally the following Conclusion and Recommendation can be made.

1. The cement used is of good quality and complied with the standard requirement
2. The fine and coarse aggregates used are as well of good quality and complied with the standard requirement.
3. The weight loss recorded at 100°C for 30minutes for combined aggregates concrete is 1.08% while that of single aggregates (20mm) concrete at the same heating condition is 1.8% (Fig. 4.5)
4. The amount of weight loss recorded for 28days old concrete in fig.4.5 and 4.20 at exposure period of 30 and 120minutes are 1.0% and 4.0 % respectively.
5. The critical temperature and exposure time beyond which the residual compressive strength drops below 75% of normal strength was found to be 600°C . and above
6. Rapid increase in residual compressive strength amounting to 18% above the strength of the unheated specimens was recorded at 100°C . This increase was attributed due to higher acceleration of hydration and drying of concrete at high temperature
7. The retained compressive strength at 300°C varied as (70%, 60%, 49% and 43%) below the original strength at exposure period of 30, 60, 90 and 120minutes for 3days aged and 93% , 99%, 83% and 75% for fully matured concrete (28 days)

8. The residual compressive strength declined extensively to about 10-0% when Concrete specimens get heated to 900°C ,
9. The flexural strength for reinforced concrete beams heated to 600°C for 60 and 120minutes were less than that for reference beams by about 25% and 32% respectively.
10. Some spalling and hair cracking was observed in the beams and concrete cubes exposed to fire for 120minutes at 600°C and 900°C respectively

5.2 RECOMMENDATION:

At the end of this work, it is recommended that

Incorporation of pozzalanas as partial replacement to cement to see if there will be any improvement in terms of fire resistance of concrete will be a welcome idea

CHAPTER SIX:

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APPENDICES



APPENDIX A1: Sourcing of Coarse Aggregates at Mother Cat quarry in Rimi L. G. Katsina State, 2010



Appendix A2



Appendix A3



APPENDIX A4



Appendix A5

Appendices A2 to A5 Tensile Test at Kaduna Polytechnic, Sept. 2010



Appendix A-6: Setting of moulds for casting



Appendix A-7: Casting of specimens in progress



Appendix A-8: Labelling of beam specimens



Appendix A-9: Curing of specimens



Appendix A-10: Loading of specimens in furnace



Appendix A-11: Wheighing of specimens



Appendix A-12: Crushing of specimens



Appendix A-13: Flexural test



Appendix A-14: Failed beam specimens



Appendix A-15: Failed cube specimens



Appendix 16



Appendix 17



Appendix 18



Appendix 19



Appendix 20

Appendix A-21 Raw data for *Normal consistency*

TRIAL	AMOUNT OF CEMENT	AMOUNT OF WATER	DEAPTH OF PENETRATION
1	500g	150mm ³	16mm
2	500g	155mm ³	9mm
3	500g	160mm ³	5mm

Appendix A-22Raw data for setting time of cement

Time	Penetration
5 min	0
20 min	1mm
35 min	3 mm
55 min	4 mm
70 min	5 mm

Appendix A-23: DETERMINATION OF CEMENT SOUNDNESS

OPC sample	Mould No	Wt of sample (g)	Dist. b/4 boiling	Dist. after boiling	Expansion (mm)	Average Expansion(mm)
A	1	400	17.2	19.5	2.3, 2.1 2.5	2.3
	2	400	11.1	13.2		
	3	400	9.9	12.4		
B	1	400	10.0	13.7	3.7, 2.0 2.3	2.65
	2	400	16.8	15.2		
	3	400	19.0	21.25		
C	1	400	10.0	12.4	2.4, 2.3 2.6	2.4
	2	400	8.0	10.3		
	3	400	13.2	15.8		

Appendix A-24: sieve analysis of 10mm coarse aggregate

Sieve size mm	Weight retained (g)	% Retained	% passing
14.00	-	-	100
10.00	93.3	7.8	92.2
5.00	936.8	78.1	21.9
12.36	1175.6	98.0	2.0

Appendix A-25: Sieve analysis of 14mm coarse aggregates

Sieve size mm	Weight retained (g)	% Retained	% passing
20.00	-	-	100
14.00	139.5	9.3	90.7
10.00	84.1	53.6	46.4
5.00	1440.4	96.0	24.0

Appendix A-26: Sieve analysis of 20mm coarse aggregates

Sieve size mm	Weight retained (g)	% Retained	% passing
37.50	-	-	100
20,00	154.5	7.7	92.3
14.00	1020.1	51.0	49.0
10.00	1635.3	81.8	18.2
5.00	1935.8	96.8	3.2

Appendix A-27 Sieve analysis of combined coarse aggregates

Sieve size Mm	20mm		14mm		10mm	
	%passing	40%	%passing	30%	%passing	30%
37.50	100	40	100	30	100	30
20,00	92.3	36.9	100	30	100	30
14.00	49.0	19.6	90.7	27.2	100	30
10.00	18.2	7.3	46.4	13.9	92.2	27.7
5.00	3.2	1.3	4.0	1.2	21.9	6.6

Appendix A-28: Sieve analysis of fine aggregates.

Total weight of dry sample before test=800grms of river sand				
Sieve size mm	Weight retained (g)	% Retained	% passing	Cum.% passing
10.00	-		100	100
5,00	13.4	1.7	98.3	198.3
2.36	44.7	5.6	94.4	292.7
1.18	116.5	14.6	85.4	378.1
0.600	206.7	25.8	74.2	452.3
0.300	436.7	54.6	45.4	526.5
0.150	682.4	85.3	14.7	541.2
0.075	760.5	95.1	4.9	546.1

Appendix A-29: Sieve analysis of combined “coarse and fine aggregates”

Sieve size Mm	20.00mm		14.00mm		10.00mm		River sand		Combined aggregates
	%passing	24%	%passing	18%	%passing	18%	%passing	40%	
37.5	100	24	100	18	100	18	100	40	100
20.0	92.3	22.2	100	18	100	18	100	40	98.2
14.0	3.2	0.8	4.0	0.7	21.9	3.9	98.3	39.3	44.7
10.0	0.0	0	0	0	0	0	74.2	29.7	29.7
5.00	0.0	0	0	0	0	0	14.7	5.9	5.9

Appendix A-30 - Specific Gravity of 20.0mm coarse aggregates

Wt. of Bottle + Sample + Water W_3 (g)	1531.2	1473.8	1472.7
Wt. of Bottle + Sample W_2 (g)	901.6	811.0	809.3
Wt. of Bottle full of water W_4 (g)	1244.5	1244.9	1244.9
Wt. of Bottle (g)	444.2	444.2	444.2
Wt. of Sample used(g)	457.4	366.8	365.1
Wt. of water in full bottle $W_2.W_1$ (g)	800.3	800.7	880.7
Wt. of water used $W_3.W_2$ (g)	629.6	662.8	663.4
Volume of Sample $(W_4.W_1) - (W_3.W_2)$ (g)	170.7	137.9	137.3
Specific gravity $G_s =$ —————	2.680	2.660	2.659
Average Value G_s	2.660		

Appendix A-31: -Specific Gravity of 14.0mm coarse aggregates obtained from the Mother Cat Quarry

Wt. of Bottle + Sample + Water W_3 (g)	1471.8	1415.0	1370.0
Wt. of Bottle + Sample W_2 (g)	808.2	716.3	6434.9
Wt. of Bottle full of water W_4 (g)	1245.1	1244.3	1244.3
Wt. of Bottle (g)	444.2	444.2	444.3
Wt. of Sample used (g)	364.0	272.1	200.6
Wt. of water in full bottle W_2-W_{1v} (g)	800.9	800.0	880.0
Wt. of water used W_3-W_2 (g)	663.6	698.7	725.1
Volume of Sample $(W_4-W_1) - (W_3-W_2)$ (g)	137.3	101.3	74.9
Specific gravity $G_s =$ —————	2.651	2.680	2.678
Average Value G_s	2.672		

Appendix A-32: Specific Gravity of 10.0mm coarse aggregates

Wt. of Bottle + Sample + Water W_3 (g)	1483.8	1453.6	1407.1
Wt. of Bottle + Sample W_2 (g)	826.7	779.3	705.1
Wt. of Bottle full of water W_4 (g)	1244.3	1244.5	1244.2
Wt. of Bottle (g)	444.2	444.2	444.2
Wt. of Sample used (g)	382.5	335.1	260.9
Wt. of water in full bottle $W_4 - W_{1v}$ (g)	800.1	800.3	880.3
Wt. of water used $W_3 - W_2$ (g)	657.1	674.3	702.0
Volume of Sample $(W_4 - W_1) - (W_3 - W_2)$ (g)	143.0	126.0	98.3
Specific gravity $G_s = \frac{W_3 - W_2}{W_4 - W_1}$	2.672	2.660	2.654
Average Value G_s	2.663		

Appendix A-33;Specific Gravity of Sharp sand (river sand)

Wt. of Bottle + Sample + Water W_3 (g)	1423.2	1432.9	1414.1
Wt. of Bottle + Sample W_2 (g)	733.3	748.9	718.5
Wt. of Bottle full of water W_4 (g)	1244.7	1245.2	1244.3
Wt. of Bottle (g)	444.2	444.2	444.2
Wt. of Sample used (g)	289.1	304.7	274.3
Wt. of water in full bottle W_2-W_{1v} (g)	800.5	800.9	880.1
Wt. of water used W_3-W_2 (g)	689.9	684.0	695.6
Volume of Sample $(W_4-W_1) - (W_3-W_2)$ (g)	110.6	116.9	104.5
Specific gravity $G_s = \frac{W_3-W_2}{W_4-W_1}$	2.614	2.607	2.625
Average Value G_s	2.615		

Appendix A-34 : Specific Gravity of Stone dust

Wt. of Bottle + Sample + Water W_3 (g)	1449.9	1430.5	1372.1
Wt. of Bottle + Sample W_2 (g)	773.9	740.1	648.7
Wt. of Bottle full of water W_4 (g)	1245.1	1245.2	1244.0
Wt. of Bottle (g)	444.2	444.2	444.2
Wt. of Sample used (g)	329.4	295.9	204.5
Wt. of water in full bottle $W_2 - W_{1v}$ (g)	800.9	801.0	799.8
Wt. of water used $W_3 - W_2$ (g)	676.3	690.4	723.4
Volume of Sample $(W_4 - W_1) - (W_3 - W_2)$ (g)	124.6	110.6	76.4
Specific gravity $G_s = \frac{\text{Wt. of Sample}}{\text{Volume of Sample}}$	2.644	2.675	2.677
Average Value G_s	2.665		

APPENDIX A-35: DETERMINATION OF AGGREGATE CRUSHING VALUE

Test NO	Sample A	Sample B	Average	
1	2419.5	428.7	17.7%	18.7
2	2787.3	581.2	20.9%	
3	2754.0	482.3	17.5%	

Where “A” = The weight of surface Dry sample

“B” =The weight of the fraction passing 2.36mm test sieve

Hence, percentage fine = $\frac{B}{A} \times 100$

Appendix A-36_{Test 1}: Determination of Water Absorption

Test NO 1	Sample A	Sample B
Size of Aggregat 20mm	Wt. 1 = 468.9	Wt1 = 463.6
	Wt.2 =470.7	Wt.2 = 465.6
	$\frac{1.8}{W1} \times 100$ = 0.4%	$\frac{2.0}{W1} \times 100$ = 0.4%

Appendix A-37_{Test2}: Determination of Water Absorption

Test NO 2	Sample A	Sample B
14mm aggregate	Wt. 1 = 220.8	Wt1 = 253.8
	Wt.2 =221.9	Wt.2 = 255.1
	$\frac{1.1}{W1} \times 100$ $= 0.5\%$	$\frac{21.3}{W1} \times 100$ $= 0.5\%$

Appendix A38_{Test3} :Determination of Water Absorption

Test NO 3	Sample A	Sample B
Size of Aggregates (10mm)	Wt. 1 = 408.5	Wt1 = 463.6
	Wt.2 = 409.3	Wt.2 = 465.6
	$\frac{0.8}{W1} \times 100$ $= 0.2\%$	$\frac{0.9}{W1} \times 100$ $= 0.2\%$

Appendix A-39_{Test4}: Determination of Water Absorption

Test NO 4	Sample A	Sample B
Size of Aggregates. (Sharp sand)	Wt. 1 = 1168,5	Wt1 = 816.5
	Wt.2 = 1169.0	Wt.2 = 820.7
	6.5 / W1x 100 = 0.6%	2.4.2W1 x100 = 0.5%
	Average Value = 0.6%	

APPENDIX A-40₁: Normal Elastic Modulus at 60mins exposure period.

Elastic Modulus at 60mins exposure period				
S/N	Temperature ⁰ c	Elastic Modulus N/mm ²		
1	0	2436	2780	2920
2	100	2268	2420	2568
3	300	1932	2031	2100
4	600	1011	1293	1432

APPENDIX A-41₂: Normal Elastic Modulus at 120mins exposure period.

Elastic Modulus at 120mins exposure period				
S/N	Temperature ⁰ c	Elastic Modulus N/mm ²		
1	0	2436	2780	2920
2	100	2268	2420	2568
3	300	1932	2031	2100
	600	1011	1293	1432

APPENDIX A-42₃: % of Normal Elastic Modulus at 60mins exposure period

% of normal Elastic Modulus at 60mins exposure period				
S/N	Temperature ⁰ c	Elastic Modulus N/mm ²		
1	0	100	100	100
2	100	93.1	87.0	88.0
3	300	79.3	73	72.0
4	600	41.5	46.3	49.0

APPENDIX A-43₄ Normal Elastic Modulus at 120mins exposure period

% of Normal Elastic Modulus at 120mins exposure period				
S/N	Temperature ⁰ c	Elastic Modulus N/mm ²		
1	0	100	100	100
2	100	71.4	85.4	88.0
3	300	72	69	75
4	600	45	48	49

Appendix A-44: Control Specimens: Compressive Strength of Control Specimens

Spec. Mark	Date Cast	Spec. size (mm)	Wt. b/4 Curinig	Wt. after Curinig	Wt. difference	Date of Test	Failure Load(K N)	Compressive Strength(N/m ²)	Mean Strengt h
A ₁	2/9/2010	150x150x150	8.7	8.5	0.2	3/9/2010	310	13.2	13.5
A ₂			8.8	8.6	0.2		315	14.0	
A ₃			8.6	8.5	0.1		310	13.7	
B ₁	2/9/2010	150x150x150	8.9	8.6	0.3	5/9/2010	345	15.3	15.3
B ₂			8.8	8.5	0.3		350	15.5	
B ₃			8.7	8.6	0.4		340	15.2	
C ₁	2/9/2010	150x150x150	8.6	8.5	0.1	9/9/2010	470	20.8	20.8
C ₁			8.7	8.5	0.2		475	21.1	
C ₁			8.5	8.4	0.1		460	20.4	
D ₁	2/9/2010	150x150x150	8.9	8.7	0.2	16/9/2010	550	24.4	24.2
D ₂			8.7	8.5	0.2		555	24.6	
D ₃			8.5	8.4	0.1		549	24.1	
E ₁	2/9/2010	150x150x150	8.9	8.7	0.2	30/9/2010	710	31.5	31.6
E ₂			8.7	8.5	0.2		715	31.7	
E ₃			8.5	8.4	0.1		700	31.1	

Appendix A-45-100°C @ 30min: Residual Compressive Strength of Combined Aggregates (RAW DATA)

Spec. Mark	Date Cast	Spec. size (mm)	Wt. b/4 heating	Wt. after heating	Wt. difference	Temperature °C	Date of Test	Failure Load (KN)	Compressive Strength (N/mm ²)	Mean Strength
F ₁	3/9/2010	150x150x150	8.9	8.6	0.3	100	4/9/2010	120	5.1	5.1
F ₂			8.8	8.6	0.2			121	5.2	
F ₃			9.0	8.8	0.2			120	5.1	
G ₁	3/9/2010	150x150x150	8.9	8.6	0.3		6/9/2010	345	15.3	15.3
G ₂			8.8	8.5	0.3			350	15.5	
G ₃			8.7	8.6	0.4			340	15.2	
H ₁	3/9/2010	150x150x150	8.6	8.5	0.1		10/9/2010	430	19.1	19.1
H ₂			8.7	8.5	0.2			435	19.3	
H ₃			8.5	8.4	0.1			430	19.1	
I ₁	3/9/2010	150x150x150	8.9	8.7	0.2		17/9/2010	550	24.4	24.2
I ₂			8.7	8.5	0.2			555	24.6	
I ₃			8.5	8.4	0.1			549	24.1	
J ₁	3/9/2010	150x150x150	8.9	8.7	0.2		31/9/2010	710	31.5	31.6
J ₂			8.7	8.5	0.2			715	31.7	
J ₃			8.5	8.4	0.1			700	31.1	

Appendix A-46-100°C @ 60min: Residual Compressive Strength of Combined Aggregates

Spec. Mark	Date Cast	Spec. size (mm)	Wt. b/4 heating	Wt. after heating	Wt. difference	Temperature °C	Date of Test	Failure Load (KN)	Compressive Strength (N/mm ²)	Mean Strength
k ₁	4/9/2010	150x150x150	8.9	8.6	0.3	100	5/9/2010	125	5.5	5.2
k ₂			8.8	8.6	0.2			118	5.2	
k ₃			9.0	8.8	0.2			115	5.1	
l ₁	4/9/2010	150x150x150	8.9	8.6	0.3		7/9/2010	345	15.3	15.3
l ₂			8.8	8.5	0.3			350	15.5	
l ₃			8.7	8.6	0.4			340	15.2	
m ₁	4/9/2010	150x150x150	8.6	8.5	0.1		11/9/2010	440	19.1	19.6
m ₂			8.7	8.5	0.2			435	19.3	
m ₃			8.5	8.4	0.1			440	19.1	
l ₁	4/9/2010	150x150x150	8.9	8.7	0.2		18/9/2010	545	24.4	22.2
l ₂			8.7	8.5	0.2			545	24.6	
l ₃			8.5	8.4	0.1			540	24.1	
n ₁	4/9/2010	150x150x150	8.9	8.7	0.2		33/9/2010	705	31.5	30.4
n ₂			8.7	8.5	0.2			710	31.7	
n ₃			8.5	8.4	0.1			700	31.1	

Appendix A-47:100°C @ 90min : Residual Compressive Strength of Combined Aggregates

Spec. Mark	Date Cast	Spec. size (mm)	Wt. b/4 heating	Wt. after heating	Wt. difference	Temperature °C	Date of Test	Failure Load (KN)	Compressive Strength (N/mm ²)	Mean Strength
o ₁	5/9/2010	150x150x150	8.9	8.6	0.3	100	7/9/2010	125	5.5	5.2
o ₂			8.8	8.6	0.2			118	5.2	
o ₃			9.0	8.8	0.2			115	5.1	
p ₁	5/9/2010	150x150x150	8.9	8.6	0.3		9/9/2010	345	15.3	15.0
p ₂			8.8	8.5	0.3			340	15.0	
p ₃			8.7	8.6	0.4			340	15.0	
q ₁	5/9/2010	150x150x150	8.6	8.5	0.1		12/9/2010	400	17.7	17.1
q ₂			8.7	8.5	0.2			385	16.8	
q ₃			8.5	8.4	0.1			390	17.0	
R ₁	5/9/2010	150x150x150	8.9	8.7	0.2		18/9/2010	500	22.2	21.1
R ₂			8.7	8.5	0.2			505	22.4	
R ₃			8.5	8.4	0.1			470	20.4	
S ₁	5/9/2010	150x150x150	8.9	8.7	0.2		33/9/2010	705	31.5	30.6
S ₂			8.7	8.5	0.2			710	31.7	
S ₃			8.5	8.4	0.1			700	31.1	

Appendix A-48-100°C @ 120min Residual Compressive Strength of Combined Aggregates

Spec. Mark	Date Cast	Spec. size (mm)	Wt. b/4 heating	Wt. after heating	Wt. difference	Temperature °C	Date of Test	Failure Load (KN)	Compressive Strength (N/mm ²)	Mean Strength
T ₁	8/9/2010	150x150x150	8.8	8.6	0.2	100	10/9/2010	120	5.5	5.0
T ₂			9.0	8.6	0.4			122	5.2	
T ₃			9.0	8.8	0.2			120	5.1	
U ₁	8/9/2010	150x150x150	8.9	8.6	0.3		13/9/2010	325	14.3	12.3
U ₂			8.8	8.5	0.3			320	13.5	
U ₃			8.7	8.6	0.4			310	11.7	
V ₁	8/9/2010	150x150x150	8.6	8.5	0.1		21/9/2010	430	19.1	17.6
V ₂			8.7	8.5	0.2			425	16.3	
V ₃			8.5	8.4	0.1			420	16.1.	
W ₁	8/9/2010	150x150x150	8.9	8.7	0.2		29/9/2010	535	23.4	20.2
W ₂			8.7	8.5	0.2			510	22.6	
W ₃			8.5	8.4	0.1			400	17.1	
X ₁	8/9/2010	150x150x150	8.9	8.7	0.2		14/10/2010	555	24.5	25.6
X ₂			8.7	8.5	0.2			610	27.1	
X ₃			8.5	8.4	0.1			600	26.1	

Appendix A-49-300°C @ 60min: Residual Compressive Strength of Combined Aggregates

Spec. Mark	Date Cast	Spec. size (mm)	Wt. b/4 heating	Wt. after heating	Wt. difference	Temperature °C	Date of Test	Failure Load (KN)	Compressive Strength (N/mm ²)	Mean Strength
Y ₁	10/9/2010	150x150x150	9.0	8.6	0.4	300	2/9/2010	135	5.8	5.5
Y ₂			8.7	8.6	0.1			128	5.6	
Y ₃			8.8	8.7	0.1			115	5.1	
Z ₁	10/9/2010	150x150x150	8.8	8.6	0.2		14/9/2010	305	13.5	12.3
Z ₂			8.8	8.5	0.3			300	13.3	
Z ₃			8.9	8.7	0.2			260	11.5	
AA ₁	10/9/2010	150x150x150	8.8	8.5	0.1		22/9/2010	400	17.7	18.0
AA ₂			8.7	8.5	0.2			415	18.4	
AA ₃			8.5	8.4	0.1			410	17.9	
BB ₁	10/9/2010	150x150x150	8.9	8.7	0.2		30/9/2010	545	24.4	22.2
BB ₂			8.7	8.5	0.2			505	22.4	
BB ₃			8.5	8.4	0.1			440	19.5	
CC ₁	10/9/2010	150x150x150	8.9	8.7	0.2		16/10/2010	705	31.5	29.3
CC ₂			8.7	8.5	0.2			630	28.7	
CC ₃			8.5	8.4	0.1			600	27.7	

Appendix A-50-300°C @ 90min: Residual Compressive Strength of Combined Aggregates

Spec. Mark	Date Cast	Spec. size (mm)	Wt. b/4 heating	Wt. after heating	Wt. difference	Temperature ⁰ C	Date of Test	Failure Load(KN)	Compressive Strength(N/mm ²)	Mean Strength
DD ₁ DD ₂ DD ₃	12/9/2010	150x150x150	8.9 8.8. 9.0	8.6 8.6 8.8	0.3 0.2 0.2	300	1-day 14/9/2010	65 80 60	2.7 3.5 2.6	2.9
EE ₁ EE ₂ EE ₃	12/9/2010	150x150x150	8.9 8.8 8.7	8.6 8.5 8.6	0.3 0.3 0.4		3-days 20/9/2010	225 210 240	11.3 9.3 10.6	10.3
FF ₁ FF ₂ FF ₃	12/9/2010	150x150x150	8.6 8.7 8.5	8.5 8.5 8.4	0.1 0.2 0.1		7-days 28/9/2010	300 400 340	13.3 17.7 15.1.	15.4
GG ₁ GG ₂ GG ₃	12/9/2010	150x150x150	8.9 8.7 8.5	8.7 8.5 8.4	0.2 0.2 0.1		14-days 7/10/2010	500 525 440	22.2 23.4 19.5	22.2
HH ₁ HH ₂ HH ₃	12/9/2010	150x150x150	8.9 8.7 8.5	8.7 8.5 8.4	0.2 0.2 0.1		28-days 22/10/2010	605 650 600	26.5 28.8 26.7	27.4

Appendix A-51-300°C @ 9\120min: Residual Compressive Strength of Combined Aggregates

Spec. Mark	Date Cast	Spec. size (mm)	Wt. b/4 heating	Wt. after heating	Wt. difference	Temperature ⁰ C	Date of Test	Failure Load(KN)	Compressive Strength(N/mm ²)	Mean Strength
II ₁	15/9/2010	150x150x150	8.9	8.6	0.3	300	1-day 17/9/2010	60	2.6	2.4
II ₂			8.8	8.6	0.2			50	2.2	
II ₃			9.0	8.8	0.2			60	2.6	
JJ ₁	15/9/2010	150x150x150	8.9	8.6	0.3		3-days 24/9/2010	205	8.9	9.3
JJ ₂			8.8	8.5	0.3			210	9.3	
JJ ₃			8.7	8.6	0.4			230	10.2	
KK ₁	15/9/2010	150x150x150	8.6	8.5	0.1		7-days 1/10/2010	300	13.3	15.4
KK ₂			8.7	8.5	0.2			400	17.7	
KK ₃			8.5	8.4	0.1			340	15.1.	
LL ₁	15/9/2010	150x150x150	8.9	8.7	0.2		14-days 9/10/2010	400	18.2	16.9
LL ₂			8.7	8.5	0.2			325	14.4	
LL ₃			8.5	8.4	0.1			310	18.2	
J	15/9/2010	150x150x150	8.9	8.7	0.2		28-days 25/10/2010	505	22.5	21.4
M			8.7	8.5	0.2			500	22.2	
M ₁			8.5	8.4	0.1			400	17.7	
M										
M ₂										
M										
M ₃										

Appendix A-52-500°C @ 30min: Residual Compressive Strength of Combined Aggregates

Spec. Mark	Date Cast	Spec. size (mm)	Wt. b/4 heating	Wt. after heating	Wt. difference	Temperature ⁰ C	Date of Test	Failure Load(KN)	Compressive Strength(N/mm ²)	Mean Strength
NN ₁ NN ₂ NN ₃	17/9 /201 0	150x150 x150	8.9 8.8 9.0	8.6 8.6 8.8	0.3 0.2 0.2	500	1-day 19/9/20 10	25 30 10	1.1 1.3 0.4	0.9
OO ₁ OO ₂ OO ₃	17/9 /201 0	150x150 x150	8.9 8.8 8.7	8.6 8.5 8.6	0.3 0.3 0.4		3-days 21/9/20 10	125 180 140	5.5 8.3 6.2	6.6
PP ₁ PP ₂ PP ₃	17/9 /201 0	150x150 x150	8.6 8.7 8.5	8.5 8.5 8.4	0.1 0.2 0.1		7-days 25/9/20 10	250 200 200	11.1 8.7 8.7	9.9
QQ ₁ QQ ₂ QQ ₃	17/9 /201 0	150x150 x150	8.9 8.7 8.5	8.7 8.5 8.4	0.2 0.2 0.1		14-days 3/10/20 10	300 255 280	13.2 11.3 12.5	12.2
RRR ₁ RR ₂ RR ₃	17/9 /201 0	150x150 x150	8.9 8.7 8.5	8.7 8.5 8.4	0.2 0.2 0.1		28-days 19/10/2 010	405 450 380	18.0 20.0 16.7	18.4

Appendix A-53-500°C @ 60min: Residual Compressive Strength of Combined Aggregates

Spec. Mark	Date Cast	Spec. size (mm)	Wt. b/4 heating	Wt. after heating	Wt. difference	Temperature °C	Date of Test	Failure Load (KN)	Compressive Strength (N/mm ²)	Mean Strength
SS ₁	18/9/2010	150x150x150	8.9	8.6	0.3	500	1-day 20/9/2010	10	0.4	0.9
SS ₂			8.8	8.6	0.2			25	1.1	
SS ₃			9.0	8.8	0.2			30	1.3	
TT ₁	18/9/2010	150x150x150	8.9	8.6	0.3		3-days 23/9/2010	125	5.5	6.4
TT ₂			8.8	8.5	0.3			170	7.5	
TT ₃			8.7	8.6	0.4			140	6.2	
UU ₁	18/9/2010	150x150x150	8.6	8.5	0.1		7-days 28/9/2010	200	8.7	9.9
UU ₂			8.7	8.5	0.2			250	11.1	
UU ₃			8.5	8.4	0.1			200	8.7	
VV ₁	18/9/2010	150x150x150	8.9	8.7	0.2		14-days 6/10/2010	300	13.2	12.2
VV ₂			8.7	8.5	0.2			255	11.3	
VV ₃			8.5	8.4	0.1			280	12.5	
WW ₁	18/9/2010	150x150x150	8.9	8.7	0.2		28-days 21/10/2010	320	14.2	14.9
WW ₂			8.7	8.5	0.2			400	17.8	
WW ₃			8.5	8.4	0.1			280	12.7	

Appendix A-54-500°C @ 90min: Residual Compressive Strength of Combined Aggregates

Spec. Mark	Date Cast	Spec. size (mm)	Wt. b/4 heating	Wt. after heating	Wt. difference	Temperature °C	Date of Test	Failure Load (KN)	Compressive Strength (N/mm ²)	Mean Strength
XX ₁	18/9/2010	150x150x150	8.9	8.6	0.3	500	1-day 20/9/2010	10	0.4	0.7
XX ₂			8.8	8.6	0.2			25	0.9	
XX ₃			9.0	8.8	0.2			30	1.3	
YY ₁	18/9/2010	150x150x150	8.9	8.6	0.3		3-days 23/9/2010	125	5.5	5.7
YY ₂			8.8	8.5	0.3			120	5.5	
YY ₃			8.7	8.6	0.4			130	5.8	
ZZ ₁	18/9/2010	150x150x150	8.6	8.5	0.1		7-days 28/9/2010	200	8.7	8.9
ZZ ₂			8.7	8.5	0.2			200	8.7	
ZZ ₃			8.5	8.4	0.1			200	8.7	
AB ₁	18/9/2010	150x150x150	8.9	8.7	0.2		14-days 6/10/2010	200	8.7	10.7
AB ₂			8.7	8.5	0.2			250	11.0	
AB ₃			8.5	8.4	0.1			280	12.5	
AC ₁	18/9/2010	150x150x150	8.9	8.7	0.2		28-days 21/10/2010	320	14.2	13.4
AC ₂			8.7	8.5	0.2			300	13.3	
AC ₃			8.5	8.4	0.1			280	12.7	

Appendix A-55-500°C @ 120min: Residual Compressive Strength of Combined Aggregates

Spec. Mark	Date Cast	Spec. size (mm)	Wt. b/4 heating	Wt. after heating	Wt. difference	Temperature °C	Date of Test	Failure Load (KN)	Compressive Strength (N/mm ²)	Mean Strength
AD ₁ AD ₂ AD ₃	18/9 /2010	150x150 x150	8.9 8.8 9.0	8.6 8.6 8.8	0.3 0.2 0.2	500	1-day 20/9/2010	10 15 20	0.4 0.6 1.1	0.7
AE ₁ G _{AE} AE ₃	18/9 /2010	150x150 x150	8.9 8.8 8.7	8.6 8.5 8.6	0.3 0.3 0.4		3-days 23/9/2010	105 100 90	4.5 4.4 4.0	4.5
AF ₁ AF ₂ AF ₃	18/9 /2010	150x150 x150	8.6 8.7 8.5	8.5 8.5 8.4	0.1 0.2 0.1		7-days 28/9/2010	100 200 120	4.5 8.8 5.3	6.2
AG ₁ AG ₂ AG ₃	18/9 /2010	150x150 x150	8.9 8.7 8.5	8.7 8.5 8.4	0.2 0.2 0.1		14-days 6/10/2010	250 200 230	11.2 8.8 10.3	10.7
AH ₁ AH ₂ AH ₃	18/9 /2010	150x150 x150	8.9 8.7 8.5	8.7 8.5 8.4	0.2 0.2 0.1		28-days 21/10/2010	320 360 280	14.2 16.8 12.7	14.1

Appendix A-56-700°C @ 30min: Residual Compressive Strength of Combined Aggregates

Spec. Mark	Date Cast	Spec. size (mm)	Wt. b/4 heating	Wt. after heating	Wt. difference	Temperature °C	Date of Test	Failure Load (KN)	Compressive Strength (N/mm ²)	Mean Strength
Al ₁	18/9/2010	150x150x150	8.9	8.6	0.3	700	1-day 20/9/2010	3	0.1	0.2
Al ₂			8.8	8.6	0.2			5	0.2	
Al ₃			9.0	8.8	0.2			6	0.2	
AJ ₁	18/9/2010	150x150x150	8.9	8.6	0.3		3-days 23/9/2010	30	1.3	1.2
AJ ₂			8.8	8.5	0.3			25	1.1	
AJ ₃			8.7	8.6	0.4			30	1.3	
AK ₁	18/9/2010	150x150x150	8.6	8.5	0.1		7-days 28/9/2010	100	4.4	5.1
AK ₂			8.7	8.5	0.2			150	6.6	
AK ₃			8.5	8.4	0.1			100	4.4	
AL ₁	18/9/2010	150x150x150	8.9	8.7	0.2		14-days 6/10/2010	200	8.8	5.6
AL ₂			8.7	8.5	0.2			95	4.2	
AL ₃			8.5	8.4	0.1			80	3.5	
AM ₁	18/9/2010	150x150x150	8.9	8.7	0.2		28-days 21/10/2010	220	9.8	12.4
AM ₂			8.7	8.5	0.2			300	13.3	
AM ₃			8.5	8.4	0.1			280	12.7	

Appendix A-57-700°C @ 60min: Residual Compressive Strength of Combined Aggregates

Spec. Mark	Date Cast	Spec. size (mm)	Wt. b/4 heating	Wt. after heating	Wt. difference	Temperature °C	Date of Test	Failure Load (KN)	Compressive Strength (N/mm ²)	Mean Strength
AN ₁ AN ₂ AN ₃	18/9 /201 0	150x150 x150	8.9 8.8 9.0	8.6 8.6 8.8	0.3 0.2 0.2	700	1-day 20/9/20 10	10 25 20	0.4 1.1 1.0	0.8
AO ₁ AO ₂ AO ₃	18/9 /201 0	150x150 x150	8.9 8.8 8.7	8.6 8.5 8.6	0.3 0.3 0.4		3-days 23/9/20 10	125 140 140	5.5 6.2 6.2	5.9
AP ₁ AP ₂ AP ₃	18/9 /201 0	150x150 x150	8.6 8.7 8.5	8.5 8.5 8.4	0.1 0.2 0.1		7-days 28/9/20 10	180 200 200	8.0 8.8 8.8	8.5
AQ ₁ AQ ₂ AQ ₃	18/9 /201 0	150x150 x150	8.9 8.7 8.5	8.7 8.5 8.4	0.2 0.2 0.1		14-days 6/10/20 10	200 205 280	8.8 8.9 12.5	10.7
AR ₁ AR ₂ AR ₃	18/9 /201 0	150x150 x150	8.9 8.7 8.5	8.7 8.5 8.4	0.2 0.2 0.1		28-days 21/10/2 010	320 300 280	14.2 15.0 12.4	13.8

Appendix A-58-700°C @ 90min: Residual Compressive Strength of Combined Aggregates

Spec Mark	Date Cast	Spec. size (mm)	Wt. b/4 heating	Wt. after heating	Wt. differe nce	Temperat ure ⁰ C	Date of Test	Failur e Load(KN)	Compressiv e Strength(N/ mm ²)	Mean Stren gth
AS ₁ AS ₂ AS ₃	18/9/ 2010	150x150 x150	8.9 8.8. 9.0	8.6 8.6 8.8	0.3 0.2 0.2	700	1-day 20/9/20 10	4.0 2.5 2.0	0.02 0.01 0.0	0.1
AT ₁ AT ₂ AT ₃	18/9/ 2010	150x150 x150	8.9 8.8 8.7	8.6 8.5 8.6	0.3 0.3 0.4		3-days 23/9/20 10	30 10 14	1.3 0.4 0.6	0.8
AU ₁ AU ₂ AU ₃	18/9/ 2010	150x150 x150	8.6 8.7 8.5	8.5 8.5 8.4	0.1 0.2 0.1		7-days 28/9/20 10	100 50 110	4.4 2.2 4.8	3.8
AV ₁ AV ₂ AV ₃	18/9/ 2010	150x150 x150	8.9 8.7 8.5	8.7 8.5 8.4	0.2 0.2 0.1		14-days 6/10/20 10	160 105 100	7.2 4.7 4.4	5.4
AW ₁ AW ₂ AW ₃	18/9/ 2010	150x150 x150	8.9 8.7 8.5	8.7 8.5 8.4	0.2 0.2 0.1		28-days 21/10/2 010	80 160 200	3.5 7.1 8.8	6.5

Appendix A-59-700°C @ 120min: Residual Compressive Strength of Combined Aggregates

Spec. Mark	Date Cast	Spec. size (mm)	Wt. b/4 heating	Wt. after heating	Wt. difference	Temperature °C	Date of Test	Failure Load (KN)	Compressive Strength (N/mm ²)	Mean Strength
AX ₁ AX ₂ AX ₃	18/9/2010	150x150x150	8.9 8.8 9.0	8.6 8.6 8.8	0.3 0.2 0.2	700	1-day 20/9/2010	4.0 2.5 2.0	0.0 0.2 0.0	0.1
AY ₁ AY ₂ AY ₃	18/9/2010	150x150x150	8.9 8.8 8.7	8.6 8.5 8.6	0.3 0.3 0.4		3-days 23/9/2010	15 30 10	0.6 1.3 0.4	0.7
AZ ₁ AZ ₂ AZ ₃	18/9/2010	150x150x150	8.6 8.7 8.5	8.5 8.5 8.4	0.1 0.2 0.1		7-days 28/9/2010	60 50 20	2.6 2.1 0.7	1.8
BA ₁ BA ₂ AB ₃	18/9/2010	150x150x150	8.9 8.7 8.5	8.7 8.5 8.4	0.2 0.2 0.1		14-days 6/10/2010	100 110 100	4.4 4.8 4.4	4.6
BB ₁ BB ₂ BB ₃	18/9/2010	150x150x150	8.9 8.7 8.5	8.7 8.5 8.4	0.2 0.2 0.1		28-days 21/10/2010	120 100 200	5.3 4.4 8.8	6.2

Appendix A-60-900°C @ 30min: Residual Compressive Strength of Combined Aggregates

Spec. Mark	Date Cast	Spec. size (mm)	Wt. b/4 heating	Wt. after heating	Wt. difference	Temperature ⁰ C	Date of Test	Failure Load(KN)	Compressive Strength(N/mm ²)	Mean Strength
BC ₁	18/9/2010	150x150x150	8.9	8.6	0.3	900	1-day 20/9/2010	5	0.0	0.0
BC ₂			8.8	8.6	0.2			5	0.0	
BC ₃			9.0	8.8	0.2			4	0.0	
BD ₁	18/9/2010	150x150x150	8.9	8.6	0.3		3-days 23/9/2010	10	0.1	0.0
BD ₂			8.8	8.5	0.3			10	0.1	
BD ₃			8.7	8.6	0.4			6	0.0	
BE ₁	18/9/2010	150x150x150	8.6	8.5	0.1		7-days 28/9/2010	20	0.7	0.4
BE ₂			8.7	8.5	0.2			15	0.6	
BE ₃			8.5	8.4	0.1			10	0.1	
BF ₁	18/9/2010	150x150x150	8.9	8.7	0.2		14-days 6/10/2010	60	2.6	2.2
BF ₂			8.7	8.5	0.2			55	2.3	
BF ₃			8.5	8.4	0.1			40	1.7	
BG ₁	18/9/2010	150x150x150	8.9	8.7	0.2		28-days 21/10/2010	100	4.4	4.4
BG ₂			8.7	8.5	0.2			120	5.3	
BG ₃			8.5	8.4	0.1			80	3.5	

A -61-900°C @ 60min.: Residual Compressive Strength of Combined Aggregates

Spec Mark	Date Cast	Spec. size (mm)	Wt. b/4 heati ng	Wt. after heati ng	Wt. differe nce	Temperat ure ⁰ C	Date of Test	Failur e Load(KN)	Compressiv e Strength(N/ mm ²)	Mean Stren gth
BH ₁ BH ₂ BH ₃	18/9/ 2010	150x150 x150	8.9 8.8 9.0	8.6 8.6 8.8	0.3 0.2 0.2	900	1-day 20/9/20 10	3 3 4	0.0 0.0 0.0	0.0
Bl ₁ Bl ₂ Bl ₃	18/9/ 2010	150x150 x150	8.9 8.8 8.7	8.6 8.5 8.6	0.3 0.3 0.4		3-days 23/9/20 10	11 10 7	0.1 0.1 0.0	0.0
BJ ₁ BJ ₂ BJ ₃	18/9/ 2010	150x150 x150	8.6 8.7 8.5	8.5 8.5 8.4	0.1 0.2 0.1		7-days 28/9/20 10	15 15 20	0.7 0.6 0.8	0.7
BK ₁ BK ₂ BK ₃	18/9/ 2010	150x150 x150	8.9 8.7 8.5	8.7 8.5 8.4	0.2 0.2 0.1		14-days 6/10/20 10	60 50 40	2.6 2.1 1.7	2.1
BL ₁ BL ₂ BL ₃	18/9/ 2010	150x150 x150	8.9 8.7 8.5	8.7 8.5 8.4	0.2 0.2 0.1		28-days 21/10/2 010	100 100 80	4.4 4.4 3.5	3.9

Appendix A-62-900°C @ 90min.: Residual Compressive Strength of Combined Aggregates

Spec. Mark	Date Cast	Spec. size (mm)	Wt. b/4 heating	Wt. after heating	Wt. difference	Temperature ⁰ C	Date of Test	Failure Load(KN)	Compressive Strength(N/mm ²)	Mean Strength
BM ₁ BM ₂ BM ₃	18/9 /201 0	150x150 x150	8.9 8.8 9.0	8.6 8.6 8.8	0.3 0.2 0.2	900	1-day 20/9/20 10	5 5 4	0.0 0.0 0.0	0.0
BN ₁ BN ₂ BN ₃	18/9 /201 0	150x150 x150	8.9 8.8 8.7	8.6 8.5 8.6	0.3 0.3 0.4		3-days 23/9/20 10	10 10 6	0.1 0.1 0.0	0.0
BO ₁ BO ₂ BO ₃	18/9 /201 0	150x150 x150	8.6 8.7 8.5	8.5 8.5 8.4	0.1 0.2 0.1		7-days 28/9/20 10	10 1 3	0.2 0.0 0.0	0.1
BP ₁ BP ₂ BP ₃	18/9 /201 0	150x150 x150	8.9 8.7 8.5	8.7 8.5 8.4	0.2 0.2 0.1		14-days 6/10/20 10	30 25 20	1.3 1.1 0.7	1.1
BQ ₁ BQ ₂ BQ ₃	18/9 /201 0	150x150 x150	8.9 8.7 8.5	8.7 8.5 8.4	0.2 0.2 0.1		28-days 21/10/2 010	50 70 80	2.2 3.1 3.2	2.8

Appendix A-63-900°C @ 120min.: Residual Compressive Strength of Combined Aggregates

Spec. Mark	Date Cast	Spec. size (mm)	Wt. b/4 heating	Wt. after heating	Wt. difference	Temperature ⁰ C	Date of Test	Failure Load(KN)	Compressive Strength(N/mm ²)	Mean Strength
BR ₁ BR ₂ BR ₃	18/9/2010	150x150x150	8.9 8.8 9.0	8.6 8.6 8.8	0.3 0.2 0.2	900	1-day 20/9/2010	2 4 3	0.0 0.0 0.0	0.0
BS ₁ BS ₂ BS ₃	18/9/2010	150x150x150	8.9 8.8 8.7	8.6 8.5 8.6	0.3 0.3 0.4		3-days 23/9/2010	5 7 6	0.1 0.1 0.0	0.0
BT ₁ BT ₂ BT ₃	18/9/2010	150x150x150	8.6 8.7 8.5	8.5 8.5 8.4	0.1 0.2 0.1		7-days 28/9/2010	20 15 10	0.7 0.6 0.1	0.4
BU ₁ BU ₂ BU ₃	18/9/2010	150x150x150	8.9 8.7 8.5	8.7 8.5 8.4	0.2 0.2 0.1		14-days 6/10/2010	20 15 30	0.6 0.6 1.3	0.9
BV ₁ BV ₂ BV ₃	18/9/2010	150x150x150	8.9 8.7 8.5	8.7 8.5 8.4	0.2 0.2 0.1		28-days 21/10/2010	50 30 30	2.2 1.3 1.3	1.6

Appendix A-64-900°C @ 30min: Residual Compressive Strength of Combined Aggregates

Spec. Mark	Date Cast	Spec. size (mm)	Wt. b/4 heating	Wt. after heating	Wt. difference	Temperature ⁰ C	Date of Test	Failure Load(KN)	Compressive Strength(N/mm ²)	Mean Strength
BW ₁ BW ₂ BW ₃	18/9/2010	150x150x150	8.9 8.8 9.0	8.6 8.6 8.8	0.3 0.2 0.2	900	1-day 20/9/2010	5 5 4	0.0 0.0 0.0	0.0
BX ₁ BX ₂ BX ₃	18/9/2010	150x150x150	8.9 8.8 8.7	8.6 8.5 8.6	0.3 0.3 0.4		3-days 23/9/2010	10 10 6	0.1 0.1 0.0	0.0
BY ₁ BY ₂ BY ₃	18/9/2010	150x150x150	8.6 8.7 8.5	8.5 8.5 8.4	0.1 0.2 0.1		7-days 28/9/2010	20 15 10	0.7 0.6 0.1	0.4
BZ ₁ BZ ₂ BZ ₃	18/9/2010	150x150x150	8.9 8.7 8.5	8.7 8.5 8.4	0.2 0.2 0.1		14-days 6/10/2010	60 55 40	2.6 2.3 1.7	2.2
CA ₁ CA ₂ CA ₃	18/9/2010	150x150x150	8.9 8.7 8.5	8.7 8.5 8.4	0.2 0.2 0.1		28-days 21/10/2010	100 120 80	4.4 5.3 3.5	4.4

Appendix A-65-900°C @ 30min.: Residual Compressive Strength of Combined Aggregates

Spec. Mark	Date Cast	Spec. size (mm)	Wt. b/4 heating	Wt. after heating	Wt. difference	Temperature ⁰ C	Date of Test	Failure Load(KN)	Compressive Strength(N/mm ²)	Mean Strength
CB ₁	18/9/2010	150x150x150	8.9	8.6	0.3	900	1-day 20/9/2010	5	0.0	0.0
CB ₂			8.8	8.6	0.2			5	0.0	
CB ₃			9.0	8.8	0.2			4	0.0	
CC ₁	18/9/2010	150x150x150	8.9	8.6	0.3		3-days 23/9/2010	10	0.1	0.0
CC ₂			8.8	8.5	0.3			10	0.1	
CC ₃			8.7	8.6	0.4			6	0.0	
CD ₁	18/9/2010	150x150x150	8.6	8.5	0.1		7-days 28/9/2010	20	0.7	0.4
CD ₂			8.7	8.5	0.2			15	0.6	
CD ₃			8.5	8.4	0.1			10	0.1	
CE ₁	18/9/2010	150x150x150	8.9	8.7	0.2		14-days 6/10/2010	60	2.6	2.2
CE ₂			8.7	8.5	0.2			55	2.3	
CE ₃			8.5	8.4	0.1			40	1.7	
CF ₁	18/9/2010	150x150x150	8.9	8.7	0.2		28-days 21/10/2010	100	4.4	4.4
CF ₂			8.7	8.5	0.2			120	5.3	
CF ₃			8.5	8.4	0.1			80	3.5	

CONTRIBUTION TO KNOWLEDGE

1. There is limited number of studies in the Literature comparing fire performance of combined aggregate Concrete and that of single size aggregate Concrete. The present study aims at contributing to the topic by comparing the fire resistance of the two different concretes experimentally.
2. Currently, Concrete Cover to reinforcement is the major design parameter for Fire in Codes. This research provided adequate information on the effect of concrete cover to reinforcement in fire
3. Experimental Database was generated on residual compressive and bending strength as raw data which can be used by researchers for further study.
4. Most of the researches on Reinforced Concrete members in Fire was Developed elsewhere, this research was conducted in Nigeria, therefore the Values obtained will be appropriate for Nigerian Situation
5. The effect of Fire exposure time on the Strength of Concrete in fire as provided by Codes was established, this will go a long way to give more guide and confident to the design community in selecting cover to reinforcement based on exposure time.