EFFECT OF FIRE ON THE PROPERTIES OF PLAIN AND REINFORCED CONCRETE

Project Report submitted in partial fulfilment of the requirement for the degree of

Bachelor of Technology

in

Civil Engineering

under the Supervision of

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CERTIFICATE

This is to certify that the work entitled, "**Effect of Fire on The Properties of Plain and Reinforced Concrete**" submitted by **VIVEK SINGH AND GAURAV KUMAR SRIVASTAVA** in partial fulfillment for the award of degree of Bachelor of Technology in Civil Engineering of Jaypee University of Information Technology has been carried out under my supervision. This work has not been submitted partially or wholly to any other University or Institute for the award of this or any other degree or diploma.

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CONTENT	PAGE NO.
Chapters and Appendices	
1. Introduction	
1.1 General	1
1.2 Damage Caused by Fire	2
2. Literature Review	
2.1 Plain Concrete Exposed to Fire	
2.1.1 Past Investigation	3
2.1.2 Effect on Microstructure	4
2.1.3 Effect on Compressive Strength	6
2.1.4 Effect on Aggregates	6
2.1.5 Cement paste and aggregate interaction	7
2.2 Reinforced Concrete Beam Exposed to Fire	
2.2.1 Past Investigation	7
2.2.2 Cracking Pattern	8
2.2.3 Flexural Strength	8
2.2.4 Stiffness	9
2.2.5 Shrinkage of RCC Beam	9
3. Research Significance	
3.1 General	10
4. Experimental Work	
4.1 Grading of Aggregate	12
4.2 Specific Gravity of Cement	12
4.3 Mix Design	12
4.4 Cube Casting	15
4.5 Beam Casting	16
4.6 Curing	17
4.7 Burning of Plain Concrete Cubes	18
4.8 Burning of Reinforced Concrete Beam	19
5. Results and Analysis	
5.1 Plain Concrete Cube	
5.1.1 Visual Observation	20
5.1.2 Compression Strength Test	22
5.1.3 Stress Strain Behaviour	25
5.1.4 Analysis of Stress Strain Behaviour	44
5.2 Reinforced Concrete Beam	
5.2.1 Visual Observation	45
5.2.2 Flexure Strength Test	48
5.2.3 Load Deflection Behaviour	50
5.2.4 Load Deflection Analysis	51
Conclusion	53
Appendices	54

ABSTRACT

In the present study, compressive strength, amount of water loss and load-deflection behaviour of plain concrete specimens under the effect of fire flame exposure and under uniform temperature are presented. Plain concrete cube specimens of 150 mm were subjected to fire flame temperatures ranging between 600-700ºC at age of 21 days for 3 hours.

Plain concrete cube specimens of 100 mm size were heated to two temperature levels of 200ºC and 600ºC for 3 hours. Cube compressive strength and loss in weight were observed and load deflection behaviour was analysed.

Past research has shown that the impact of high temperature on concrete involve identification of various changes in physical, thermal, and mechanical properties.

Research has demonstrated that changes in the strength of concrete as a function of temperature are related to concrete composition, w/c ratio, presence of additives, and time of concrete exposure to high temperature.

The increase in temperature results in water evaporation, C-S-H gel dehydration, calcium hydroxide and calcium carbonates decomposition etc. Due to those changes, concrete strength and modulus of elasticity gradually decrease. When 500^{0}C is passed, the compressive strength of concrete usually drops by 50% to 60%. The dehydration process of the C-S-H gel reduces its volume, which in turn increases the porosity of the cement matrix. It is generally agreed that when heated to between 300^{0}C and 600^{0}C concrete containing siliceous aggregates will turn red.

The effect of fire on reinforced concrete beam has been observed. The rcc beams were burnt with wood for 3 hours and then its residual flexural strength was checked. flexural strength, amount of water loss and load-deflection behaviour of reinforced concrete beam under the effect of fire flame exposure are presented. reinforced concrete beam of size 150X10X10 cm were subjected to fire flame temperatures ranging between 600-700ºC at age of 28 days for 3 hours.

LIST OF SYMBOLS

M- Machine Mixed Concrete Samples

H- Hand Mixed Concrete Samples size 15x15x15 cm

FH- Hand Mixed Concrete Samples size 10x10x10 cm

V28- Cubes cured for 28 days

V14- Cubes cured for 14 days

V07- Cubes cured for 7 Days

F28- Cubes cured for 28 days and heated in furnace

F14- Cubes cured for 14 days and heated in furnace

LIST OF TABLES

S.no.	*Title*
1	Grading of Fine Aggregates from IS 383-1970
2	Sieve Analysis of Fine Aggregates
3	Grade of Concrete
4	Environmental Exposure Conditions
5	Min. Cement Qty. and Concrete Grade, Max. w/c ratio for 20mm Nominal Maximum Size of Aggregate
6	Adjustment to Minimum Cement Contents
7	Compressive Strength Test Results
8	Assumed Standard Deviation
9	Values of t
10	Appropriate Air Content
11	Approximate Sand and Water Contents
12	Adjustment in Water Content and Sand Percentage
13A	Compressive strength of concrete cubes
13B	Compressive strength of concrete cubes
14	Compressive strength of unburnt sample M1
15	Compressive strength of burnt sample M2
16	Compressive strength of burnt sample M3
17	Compressive strength of unburnt sample H1
18	Compressive strength of burnt sample H2
19	Compressive strength of burnt sample H3
20	Compressive strength of unburnt sample FH1
21	Compressive strength of burnt sample FH2
22	Compressive strength of burnt sample FH3
23	Compressive strength of unburnt sample V281
24	Compressive strength of burnt sample V282

25	Compressive strength of burnt sample V283
26	Compressive strength of unburnt sample V141
27	Compressive strength of burnt sample V142
28	Compressive strength of burnt sample V143
29	Compressive strength of unburnt sample V071
30	Compressive strength of burnt sample V072
31	Compressive strength of burnt sample F28
32	Compressive strength of burnt sample F14
33	Flexure Strength of Beams
34	Load and Deflection of Beam

LIST OF FIGURES

S.no	*Caption*	*Page No.*
1	Casting of RCC Beams	16
2	Curing by Immersion and Wet Covering Methods	17
3	Picture depicting burning of cubes	18
4	Picture depicting burning of Beam	19
5a	Pictures showing change in surface colour	20
5b	Pictures showing change in colour of concrete	21
5c	Pictures showing surface cracks	22
6a	Stress-Strain Graph of unburnt sample M1	25
6b	Stress-Strain Graph of burnt sample M2	26
6c	Stress-Strain Graph of burnt sample M3	27
6d	Stress-Strain Graph of unburnt sample H1	28
6e	Stress-Strain Graph of burnt sample H2	29
6f	Stress-Strain Graph of burnt sample H3	30
6g	Stress-Strain Graph of unburnt sample FH1	31
6h	Stress-Strain Graph of burnt sample FH2	32
6i	Stress-Strain Graph of burnt sample FH3	33
6j	Stress-Strain Graph of unburnt sample V281	34
6k	Stress-Strain Graph of burnt sample V282	35
6l	Stress-Strain Graph of burnt sample V283	36
6m	Stress-Strain Graph of unburnt sample V141	37
6n	Stress-Strain Graph of burnt sample V142	38
6o	Stress-Strain Graph of burnt sample V143	39
6p	Stress-Strain Graph of unburnt sample V071	40
6q	Stress-Strain Graph of burnt sample V072	41
6r	Stress-Strain Graph of burnt sample F28	42
6s	Stress-Strain Graph of burnt sample F14	43

7	Pictures showing change in surface colour	45
8	Pictures showing change in colour of concrete	46
9	Pictures showing surface cracks	48
10a	Load Deflection Behaviour of Unburnt Beam	51
10b	Load Deflection Behaviour of Burnt Beam	51

CHAPTER 1

Introduction

1.1 General

Concrete structures could be exposed to elevated temperature conditions. Examples of such conditions are concrete foundations for launching rockets carrying spaceships, concrete structures in nuclear power stations or those accidentally exposed to fire

Normal strength concrete and high-strength concrete structures subject to fire have been studied in various aspects such as maximum temperature, heating rate, types of aggregates used and various binding materials.

When concrete is subjected to elevated temperatures, various physical (e.g., evaporation, condensation, water and vapour advection, vapour diffusion, heat conduction and advection, phase expansion), chemical (e.g., dehydration, thermo-chemical damage) and mechanical (e.g., thermo- mechanical damage, cracking, spalling) processes take place, resulting in the deterioration of the concrete.

The spalling of concrete exposed to fire has been observed under laboratory and real fire conditions. Adding polypropylene fibers to concrete mix is much more effective in minimizing spalling in HSC under fire exposure and enhances fire endurance..

A deal of information about the properties of concrete and steel after exposure to high temperatures are available. However, information about the effect of direct exposure to fire flames on properties of concrete is limited.

Cracking of concrete is perhaps its major disadvantage which results mainly from its low tensile strength and low tensile strain capacity hence concrete is considered as brittle material and lacks ductility. In general, concrete cracks when there are tensile stresses exceeding in magnitude its tensile strength.

In the structural design of buildings, in addition to the self weight and imposed loads, it is in many instances necessary to design the structure to safely resist exposure to fire. However it is usually necessary to guard against structural collapse due to fire for a given period of time.

In fire, concrete performs well as an engineered structure and as a material in its own right. It is vitally important that we create buildings and structures that protect both people and property effectively and efficiently as possible.

1.2 Damage Caused By Fire

Annual statistics on deaths caused by fires in the home and elsewhere make for unpleasant reading. In 2010-11, as many as 22,187 fire related calls were reported resulting in the death of 447 persons and injury to 2,613 persons across India.

A study of 16 industrialised nations (13 in Europe plus the USA, Canada and Japan) found that, in a typical year, the number of people killed by fires was 1 to 2 per 100,000 inhabitants and the total cost of fire damage amounted to 0.2% to 0.3% of GNP.

In the USA specifically, statistics collected by the National Fire Protection Association for the year 2000 showed that more than 4,000 deaths, over 100,000 injuries and more than $10bn of property damage were caused by fire.

UK statistics suggest that of the half a million fires per annum attended by fire fighters, about one third occur in occupied buildings and these result in around 600 fatalities (almost all of which happen in dwellings). The loss of business resulting from fires in commercial and office buildings runs into millions of pounds each year.

For example the case of fire accidents in many tunnels registered in many countries. The last one was in the tunnel between Lakhdaria and Ammal (Algeria) in February 28, 2008. Following the collision of two trains in this tunnel, 750m^3 of gas oil was burnt. The temperature has reached 1200°C. Intervention to extinct the fire was impossible and the duration of the fire was 48 hours. Dramatic damages were observed and the tunnel is closed until nowadays.

CHAPTER 2

Literature Review

2.1 Plain Concrete Exposed To Fire

Concrete does not burn. It cannot be ‘set on fire’ like other materials in a building and it does not emit any toxic fumes when affected by fire. It will also not produce smoke or drip molten particles, unlike some plastics and metals, so it does not add to the fire load. For these reasons concrete is said to have a high degree of fire resistance and, in the majority of applications, concrete can be described as virtually ‘fireproof’.

This excellent performance is due in the main to concrete’s constituent materials (i.e. cement and aggregates) which, when chemically combined within concrete, form a material that is essentially inert and, importantly for fire safety design, has a relatively poor thermal conductivity. It is this slow rate of heat transfer (conductivity) that enables concrete to act as an effective fire shield .The rate of increase of temperature through the cross section of a concrete element is relatively slow and so internal zones do not reach the same high temperatures as a surface exposed to flames.

2.1.1 Past Investigation

Concrete does not burn. It cannot be ‘set on fire’ like other materials in a building and it does not emit any toxic fumes when affected by fire. It will also not produce smoke or drip molten particles, unlike some plastics and metals, so it does not add to the fire load. For these reasons concrete is said to have a high degree of fire resistance and, in the majority of applications, concrete can be described as virtually ‘fireproof’.

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A standard ISO 834/BS 476 fire test on 160 mm wide x 300 mm deep concrete beams has shown that, after one hour of exposure on three sides, while a temperature of 600°C is reached at 16 mm from the surface, this value halves to just 300°C at 42 mm from the surface. Temperature gradient of 300 degrees in about an inch of concrete!

It has been noticed that the surface cracks became visible when the temperature reached 600 °C. The cracks were very pronounced at 800 °C and increased extremely when the temperature increased to 1000°C. The alterations produced by high temperatures are more evident when the temperature surpasses 500°C. Most changes experienced by concrete at this temperature level are considered irreversible. The range between 400 and 800°C was critical to the strength loss. It was observed that at a temperature over 600°C, all tested concretes suffered deterioration and only a small part of the initial strength was left, ranging from 7% to 25% for all mixtures.

Only few studies have evoked the combined effect of high temperature and heating time on residual strength of concrete. This subject needs more investigation that will be beneficial in engineering practice. Although the assessment of the degree of deterioration of the concrete structure after exposure to high temperatures can help engineers decide how a structure can be repaired, aesthetic damage is generally easy to repair while functional impairments are more profound and may require partial or total repair or replacement, depending on their severity.

2.1.2 Effect on Microstructure

The heating of cement paste results in drying. Water gradually evaporates from the material. The order in which water is removed from heated concrete depends on the energy that binds the water and the solid. Thus, free water evaporates first, followed by capillary water, and finally by physically bound water. The process of removing water that is chemically bound with cement hydrates is the last to be initiated.

The mechanical properties of cement paste are strongly affected by chemical bonds and cohesion forces between sheets of calcium silicate hydrate (C-S-H) gel. It is assumed that approximately 50% of cement paste strength comes from cohesion forces (important C-S-H gelsheet area); therefore, the evaporation of water between C-S-H gel sheets strongly affects the mechanical properties of the cement paste..

However, when the cement paste is heated in moist sealed conditions, hydrothermal reactions may take place. This phenomenon, called internal autoclaving may occur in large members where, due to heating, moisture is transformed into water vapour. In these conditions chemical and physical changes may take place. The process of simultaneously exposing the material to high pressures and temperature is a well-known technology in the prefabrication of concrete. This may well activate changes in the microstructure of hydrates and often increases cement paste strength.

The nature of the phase changes will depend upon the mineralogical composition of the cement, its C/S ratio (moles of lime per mol of silica; CaO/SiO2), the amount of fine particles (quartz or silica fume), and the temperature and pressure levels that have been reached. Heating the cement paste with a C/S ratio around 1.5 to temperature above 100^{o}C produces several forms of calcium silicates, in general highly porous and weak. When the C/S ratio is close to 1.0 and the temperature is above 150^{o}C, a 1.5 to 1.0 tobemorite gel can form. At temperature between 180^{o}C and 200^{o}C, other silicates such as xonolite and hillebrandite may be formed.

Recently, microstructural changes of heated cement pastes have been studied by neutron diffraction [7]. This research demonstrates the temperature at which the main products of hydration of Portland cement, including portlandite, ettringite, calcite, lime, larnite, and hydrated calcium silicate (C-S-H gel), are present. During heating, ettringite decomposes first even before the temperature reaches 100^{o}C. C-S-H gel dehydration is progressive and takes place from the very beginning of material heating. It is worth noting that the structure of the cement paste is partially damaged due to dehydration at the temperature of 105^{o}C, which is standard for the drying of materials.

The dehydration process of the C-S-H gel reduces its volume, which in turn increases the porosity of the cement matrix. Moreover, during heating, the cement paste experiences a slight expansion up to temperature of approximately 200^{o}C although the intense shrinkage begins as soon as this temperature is exceeded. This significantly contributes to the porosity evolution of the cement paste. Due to heating total pore volume increases as does the average pore size.

2.1.3 Effect on Compressive Strength

The increase in temperature results in water evaporation, C-S-H gel dehydration, calcium hydroxide and Calcium aluminates decomposition etc. Along with the increase in temperature, changes in the aggregate take place. Due to those changes, concrete strength and modulus of elasticity gradually decreases, and when the temperature exceeds temperature of 300^{o}C, the decline in strength becomes more rapid. When the 500^{o}C threshold is passed, the compressive strength of concrete usually drops by 50% to 60%, and the concrete is considered fully damaged.

2.1.4 Effect on Aggregates

The carbonate stones (limestone and dolomite) are stable up to 600^{0}C. At higher temperature, carbonate aggregate decomposes into CaO and CO^2 (700^{0}C). It leads to 44% expansion upon hydration while cooling down.

To a large extent, damage to concrete is caused by cracking, which occurs arising due to mismatched thermal strains between the coarse aggregates and the matrix. It is generally agreed that when heated to between 300^{0}C and 600^{0}C concrete containing siliceous aggregates will turn red.

At higher temperature, carbonate aggregate decomposes into CaO and CO2 (700◦C). The polymineralic stones may be prone to the disintegration that results from the thermal incompatibility of its components. For those stones differences in thermal strain can cause inter-crystalline stresses and failure.

The further heating of aggregate leads to its melting. The melting temperature varies along the mineralogical composition. For most igneous rocks it is above 1000^{o}C. The melting temperature of granites is 1210–1250^{o}C, while basalts melt at 1050^{o}C, which is accompanied by gas release and expansion.

2.1.5 Cement Paste and Aggregate Interaction

The heating of concrete makes its aggregate volume grow, and at the same time it causes the contraction of the cement paste which surrounds it. As a result, the cement paste-aggregate bond is the weakest point in heated cementitious material.

To a large extent, damage to concrete is caused by cracking, which occurs arising due to mismatched thermal strains between the coarse aggregates and the matrix. Also, cracks passing through siliceous aggregate are present, indicating the tendency of some siliceous aggregates to break up at 350°C.

2.2 Reinforced Concrete Beam Exposed to Fire

The behaviour of reinforced concrete structure exposed to fire depends on the thermal properties of steel and concrete, strength and stiffness properties of the concrete and steel at elevated temperatures, and on the ability of the structure to redistribute internal forces.

2.2.1 Past investigation

Asa´ad in (1987) studied the behaviour of structural reinforced concrete beam specimens subjected to elevated temperature. Four type of reinforced concrete samples were used. Singly and doubly reinforced concrete beams having the dimension of (100×100×1100 mm) were used. The double reinforced with both tension and compression steel, while the singly reinforced beams with tension steel only. Continuous beams (100×150×1300mm) and structure frame with outer dimensions of (900×150×1300 mm) and structure frame with outer dimensions of (900×750mm) were used.

The frame had a cross-section of (100×150mm) for the beam and (100×100mm) for the column. The specimens were subjected to temperatures of (150, 300, 600, 750 and 900 °C) at the ages of 30 and 90 days and tested the flexure after cooling. The research found that both flexural strength and stiffness decreased with the temperature increase. He also noticed that the use of top reinforcement had limited this decrease. Moreover, he observed that the increase in temperature led an increase in magnitude of moment redistribution in continues beams.

It was noticed that the load-deflection relationship of reinforced concrete beams exposed to fire flame is more levelled representing softer load-deflection behavior than that of the control beams. Also, it was found that both the ultimate resisted load and moment carrying capacity decrease remarkably after fire exposure. This can be attributed to the early cracking and lower modulus of elasticity.

Al-Abdaly found that drying shrinkage strains are remarkably less in high strength concrete than that in normal strength concrete. She also found that restrained high strength members reveal significant retardation in the commencement of drying shrinkage cracking. The behaviour of reinforced concrete structures exposed to fire depends on the thermal properties, strength and stiffness properties of concrete and steel at elevated temperatures, and on the ability of the structure to redistribute internal forces during the course of the fire studied by Purkiss.

2.2.2 Cracking Pattern

After the beams were subjected to fire flame, two types of cracks were developed; the first was thermal cracks appearing in a random fashion all over the surface. They originated from top or bottom edges and terminated near the mid-depth of the beam. The pattern of fine cracks was consistent with the release of moisture being greater in the outer surface than in the interior core resulting in differential shrinkage. The second type of cracks was flexural tensile cracks due to the three point loading.

2.2.3 Flexure Strength

Based on the results of this research, the flexure strength of concrete was affected adversely by fire flame and the degree of damage increases when the fire temperature and/or period of exposure were raised. For the fire temperatures and periods of exposure investigated, the residual flexure strength ranged between (70-78%) at 400 °C and (59-65%) at 550 °C burning temperature.

It can be indicated from the results that the ultimate load capacity of the beams is adversely influenced by the fire flame exposure and this deleterious effect decreases the ultimate load capacity by about 15-37%.

2.2.4 Stiffness

Also the maximum deflection at ultimate load increases by about 30% which shows clearly reduced stiffness behaviour. This reduction in the stiffness of the reinforced beam specimens subjected to fire flame can be attributed to the following two parameters; the first is the reduction in the moment of inertia of the section (section modulus) I_{eff} due to cracking initiating due to the effect of fire, whereas, the second is the reduction in the modulus of elasticity (E) of concrete and steel.

The experimental results clearly indicated that the temperature near the surface to fire is higher and decreases towards the top fiber of the beam thickness. Similar behaviour was observed by other investigators.

2.2.5 Shrinkage of RCC Beams

Neville (1995) reported that the given workability, which approximately means a concrete water content, shrinkage is unaffected by an increase in the cement content, or may even decrease, because the water/cement ratio is reduced and the concrete is therefore, better able to resist shrinkage.

Habeeb in (2000) found from the test results, that the additional shrinkages values due to heating are between (400-800)°C micro strains, and there is no significant increase in shrinkage values due to the increase of exposure time from 1.0 to 4.0 hours. Shrinkage values were not more than 10% of that at 1.0 hour exposure.

CHAPTER 3

Research Significance

3.1 General

A deal of information about the properties of concrete and steel after exposure to high temperature is available. However, information about the effect of direct exposure to fire flames on properties of concrete is limited.

Concrete structures could be exposed to elevated temperature conditions. Examples of such conditions are concrete foundations for launching rockets carrying spaceships, concrete structures in nuclear power stations or those accidentally exposed to fire

Normal strength concrete and high-strength concrete structures subject to fire have been studied in various aspects such as maximum temperature, heating rate, types of aggregates used and various binding materials.

When concrete is subjected to elevated temperatures, various physical (e.g., evaporation, condensation, water and vapour advection, vapour diffusion, heat conduction and advection, phase expansion), chemical (e.g., dehydration, thermo-chemical damage) and mechanical (e.g., thermo- mechanical damage, cracking, spalling) processes take place, resulting in the deterioration of the concrete.

The spalling of concrete exposed to fire has been observed under laboratory and real fire conditions. Adding polypropylene fibers to concrete mix is much more effective in minimizing spalling in HSC under fire exposure and enhances fire endurance.

The objective of these investigations was to determine the strength and deformation properties of concrete at elevated temperatures and to find out the causes of the changes that the material suffers in consequence of heat. The researchers exposed the concrete and mortar specimens to high temperatures in special furnaces.

There is indeed little research work about changes in concrete when subjected to heat in an open uncontrolled environment. There will be high and non-uniform temperature gradient and exposure time of concrete in direct contact with fire flames will be different. All the faces of sample will not receive equal amount of heat so there may be difference in the changes at the micro level between the specimens burnt in controlled and uncontrolled environment.

The present work is an attempt to investigate the effect of exposure of concrete to fire flame on shrinkage cracking, spalling and other mechanical properties of concrete.

CHAPTER 4

Experimental Work

4.1 Grading of Aggregate

Ideally coarse and fine aggregate should be graded in such a way as to minimise the voids. After compaction the volume of the cement paste must be more than the voids between particles. Underfilling will result in entrapped air and an unworkable mix.

An extreme example of this is a no-fines concrete where the sand fraction is minimised, the course aggregate interlocks, but nothing fills the voids. This will not protect the reinforcement from corrosion or provide a weather tight structure.

Similarly achieving a paste volume to 'just' fill the voids will result in a mix where the coarse aggregate will interlock but not necessarily in an optimum compacted state making placing difficult and leaving voids. Some overfilling of the void space between the coarse particles by the sand fraction and between the sand particles by a cement paste is necessary for workability, placeability and durability of the concrete.

According to Table 1 given in Appendix obtained from IS 383-1970 sand belongs to Zone III since the percentage of fine aggregates passing through 600µ sieve was found to be 59.47%. Readings are given in Table 2.

4.2 Specific Gravity of Cement

The specific gravity is normally defined as the ratio between the weight of a given volume of material and weight of an equal volume of water. Specific Gravity of Portland Pozzolana Cement was calculated as 3.15.

4.3 Mix Design

The process of selecting suitable ingredients of concrete and determining their relative amounts with the objective of producing a concrete of the required, strength, durability, and workability as economically as possible, is termed the concrete mix design. The proportioning of ingredient of concrete is governed by the required performance of concrete in 2 states, namely the plastic and the hardened states. If the plastic concrete is not workable, it cannot be properly placed and compacted. The property of workability, therefore, becomes of vital importance.

The Indian Standard Code for mix design is IS 10262-1982. The procedure has been illustrated with the help of design of M30 plain concrete.

Requirements:

i) Strength at 28 days is 30 MPa.

ii) The nominal maximum size of aggregate to be used should be 20mm.

iii) Workability should be 0.9 on compaction factor test.

iv) Degree of quality control should be good.

v) The concrete exposure is mild.

Test data for materials:

1) Specific Gravity of cement- 3.15
2) Specific Gravity of coarse aggregate- 2.66
3) Specific Gravity of fine aggregate- 2.66 (Zone III)
4) Water absorption for coarse aggregate- 0.7%
5) Water absorption for fine aggregate- 1.0%
6) Surface moisture content for coarse aggregate- nil
7) Moisture content for fine aggregate- 2%

Procedure:

i) Target Mean Strength- $\bar{f}_{ck} = f_{ck} + t*S$ t-risk factor S-Standard Deviation

From IS 456-2000 code, value of t = 1.65 (Tolerance: 1 in 20 samples) and S = 5

$\bar{f}_{ck} = 30 + 1.65*5 = 38.25$ MPa

ii) Selection of w/c ratio-

From the graph of IS 10262, for target mean strength 38.25 MPa, w/c ratio is 0.50.

For Mild Condition according to IS 456-2000,

Minimum Cement- 220 kg/m^3
Maximum w/c ratio- 0.6
Minimum Grade of Concrete- M20

iii) Selection of Water and Sand Content-

From the Table 4 of IS 10262, for 20mm aggregates

Percentage of entrapped air =2% for 20mm aggregates

Approximate Sand Content: 35% of total aggregate volume for Zone II sand

Water content: 186 kg

These are valid till M35 grade of concrete.

iv) Adjustments:

To reduce w/c ratio by 0.1, reduce sand content by 2%

To convert from zone III to zone II, from Table 6 of IS 10262, deduct 1.5% from 35% i.e. 33.5%

To reduce compaction factor from 0.9 to 0.8 reduce water content by 3%

Water content= 186-(0.03*186)= 180.6 kg

Cement Content= Water Content/(w/c ratio)

180.6/0.5= 361.2 kg

Cement content is greater than the minimum cement content. Hence correct.

Calculation of Fine and Coarse Aggregates-

$$V= [W+\{ C/S_c \}+ f_a/(P*S_{fa})]/1000$$

$$C_a= (1-P)/P*fa* S_{ca} /S_{fa})$$

V- Absolute Volume of Fresh Concrete= Gross Volume-Volume of entrapped air (m^3)

W- Mass of water (kg)

C- Mass of Cement (kg)

f_a- Mass of fine aggregates (kg)

C_a- Mass of coarse aggregates (kg)

P- Ratio of weight of fine aggregates to total aggregates

S_c- Specific Gravity of cement

S_{fa}- Specific Gravity of fine aggregates

S_{ca}- Specific Gravity of coarse aggregates

0.98= [180.6+361.2/3.15+ f_a/(0.335*2.66)/1000

f_a= 610.17 kg

Therefore C_a= 1220.34 kg

v) Second set of Adjustments-

Water absorbed by coarse aggregates = 0.7% of 1220.34= 8.54 kg

Water absorbed by fine aggregates = 1.0% of 610.17= 6.10 kg

But fine aggregates contain water = 2.0% of 610.17= 12.20 kg

Net Water required= 180.6+8.54+6.10-12.20 = 183.04 kg

New fine aggregates content= f_a+ 2% of f_a= 622.4 kg

Hence Amount of Ingredients for 1 m^3 of M30 concrete-

Cement- 361.2 kg

Fine Aggregates- 622.4 kg

Coarse Aggregates- 1220.34 kg

Water- 183.04 kg

4.4 Cube Casting

M30 grade plain concrete was made with the help of Mixer. According to the mix design given above we added water, cement and aggregates. All the ingredients were mixed by hand and then the mixture was poured in the concrete cube mould in three layers after tamping each layer 25 times by using tamping rod and then further compaction was done by placing concrete filled moulds on the vibrator table. After placing concrete cubes were left for 24 hrs to harden and then then they were immersed in the curing tank for curing.

Four cubes of 15X15X15 cm size were casted and have been represented by "M". Extra water was required than calculated which resulted in bleeding of cubes while vibrating. Another mix of M30 was prepared by hand mixing. Again four cubes of this stiff mixture were casted of 15X15X15 cm size and have been represented by "H". fifteen cubes (FH) of 10X10X10 cm size were also made for controlled heating.

4.5 Beam Casting

M30 grade reinforced concrete beam was made. According to the mix design given above we added water, cement and aggregates. All the ingredients were mixed by hand and then the mixture was poured in the concrete beam mould in three layers after tamping each layer 25 times by using tamping rod and then further compaction was done by placing concrete filled moulds on the vibrator table. The size of the mould was 50X10X10 cm. After placing concrete cubes were left for 24 hrs to harden and then they were immersed in the curing tank for curing.

The doubly reinforced steel mesh was inserted in the concrete beam mould prior to pouring concrete. Then compaction was done on the vibrator table. After casting concrete beams were left for 24 hrs to harden and they were immersed in the curing tank for curing.

Fig 1: Casting of RCC Beams

Two doubly reinforced beam was made of size 50X10X10 cm. out of which one was to be tested directly for compression and the other one was to be fire damaged for 3 hrs and wad then to be tested for compressive strength.

4.6 Curing

Curing is the process in which the concrete is protected from loss of moisture and kept within a reasonable temperature range. This process results in concrete with increased strength and decreased permeability. Curing is also a key player in mitigating cracks which can severely affect durability. A concrete element is expected to last a certain number of years. In order to meet this expected service life, it must be able to withstand structural loading, fatigue, weathering, abrasion, and chemical attack. The duration and type of curing plays a big role in determining the required materials necessary to achieve the high level of quality.

The concrete cubes and reinforced concrete beam were cured for 7,14,21,28 days after day of casting.

These all samples were kept for conventional curing after 24 hours of their casting for 7,14,21,28 days. The curing was done at room temperature. For a few days wet curing was also done with the help of thick pieces of cloth soaked in water.

Fig 2: Curing of Immersion and Wet Covering Methods

4.7 Burning of Plain Concrete Cubes

After 7, 14, 21, 28 days of curing cubes were left for air dry for few hours. Then concrete cube samples were wood fired. The wooden logs were set ablaze for three hours and thereafter samples were left on the site for next 20 hours.

Usually the temperature of fire flames emitted due to burning of wooden logs is around 500-600°C. Although it was best ensured that all the cubes and beam get the same amount of exposure to fire but since whole process of burning was manual there occurred uncontrolled heating, due to which some cubes were burnt more and some were less burnt.

Fig 3: Burning of Cubes

Next day one concrete cube sample was kept in muffle furnace for three hours at a temperature of 200°C. Another concrete cube sample was kept in muffle furnace for the same three hours at a high temperature of 600°C. They were let in open to cool down for next 24 hours.

4.8 Burning of Reinforced Concrete Beam

After 28 days beam were left to air dry for few hours. Then reinforced concrete beam was wood fired. The wooden logs were set ablaze for three hours and thereafter beam was left on the site for next 20 hours.

Usually the temperature of fire flames emitted due to burning of wooden logs is around 500-600°C. Although it was best ensured that all part of beam get the same amount of exposure to fire but since whole process of burning was manual there occurred uncontrolled heating, due to which some part of beam was less burnt and some part was rigorously burnt.

Fig 4:Burning of Beam

All samples were ready for the visual observations and flexure strength test to be done on the Universal Testing Machine.

CHAPTER 5

Result and Analysis

5.1 Plain Concrete Cube

5.1.1 Visual Observation

1) After direct exposure to fire flame the concrete surface turned black from grey.

   Reason: Colour change from grey to black was on account of external factors. It occurred due to incomplete combustion of wood that was used for burning.

$$2C + O_2 \rightarrow 2CO \text{ (Carbon Monoxide)}$$

Before burning

After burning

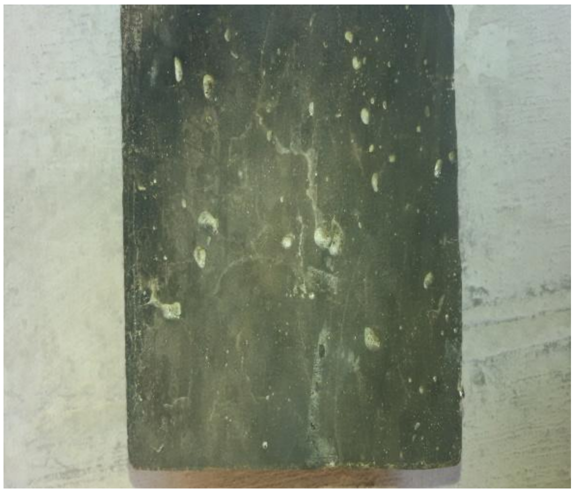


Fig 5a: Pictures Showing Change in Surface Colour

2) The concrete after burning at 600 °C turned reddish from grey colour.

Reasons: The colour change of concrete after burning from grey to reddish is due to presence of siliceous aggregate in the concrete . It is generally agreed that when heated to between 300 ◦C and 600 ◦C concrete containing siliceous aggregates will turn red; between 600◦C and 900◦C, whitish-grey; and between 900◦C and 1000◦C, a buff colour is present.

The colour change of heated concrete results principally from the gradual water removal and dehydration of the cement paste, but also transformations occurring within the aggregate. The most intense colour change, the appearance of red colouration, is observed for siliceous riverbed aggregates containing iron. This colouration is caused by the oxidation of mineral components.

While siliceous aggregates turn red when heated, the aggregates containing calcium carbonate get whitish. Due to calcination process CaCO3 turns to lime and give pale shades of white and grey.

Before burning After burning

Fig 5b: Pictures Showing Change in Colour of Concrete

3) After being exposed to direct fire flames and uniform temperature of 600 °C minor cracks developed on the surface that were visible from naked eyes only. The cube that was subjected to 200°C (FH-2) had not developed any surface cracks. The pattern of cracked developed is web pattern.

Reason: Thermal cracking occurred in the samples. It occurs due to excessive temperature difference within a concrete structure or its surroundings. This difference causes the cooler portion to contract more than the warmer portion, which restrain the contraction. Random cracks develop on surfaces. This restraint creates tensile stresses that crack the concrete as a result of this uncontrolled temperature difference across the cross-section.

There has been the loss of water while burning. It may have induced shrinkage as due to combined effect of thermal difference and shrinkage, vertical cracks from base are usually observed.

There are two types of cracks that occurred . first type of cracks are cracks arising due to thermal expansion. Thermal cracks are minor and randomly oriented depicting a pattern of web. these type of cracks occur due to difference in expansion of aggregates and cement matrix.

On exposure to temperatures aggregates and cement matrix which mainly contains product of hydration expands differently. Aggregates show greater expansion than the cement matrix. Second types of cracks that occur are shrinkage cracks. Such cracks start from edges and run perpendicular to edges. They are wider than thermal cracks.

In the sample FH2, there were no cracks found. Till the temperature of 200°C, concrete can bear the thermal stresses without cracking.

Before burning After burning

Fig 5c: Pictures Showing Surface Cracks

5.1.2 Compression Strength Test

Compression strength test was performed on both burnt and unburnt samples that were damaged by direct flame exposure and heated at uniform temperature at 200°C and 600 °C by universal testing machine.

TABLE 13A: Compressive Strength of Concrete Cubes

NAME OF SAMPLE	CONDITION	ULTIMATE STRENGTH (MPa)	% CHANGE IN STRENGTH
M1	Unburnt	19.796	
M2	Burnt	25.924	30.95
M3	Burnt	25.742	30.03
H1	Unburnt	26.053	
H2	Burnt	36.147	38.74
H3	Burnt	39.236	50.6
FH1	Unburnt	33.200	
FH2	Burnt	28.900	12.95
FH3	Burnt	25.600	22.89

TABLE 13B: Compressive Strength of Concrete Cubes

NAME OF SAMPLE	CONDITION	ULTIMATE STRENGTH MPa	% CHANGE IN STRENGTH
V281	Unburnt	38	
V282	Burnt	51.29	34.97
V283	Burnt	58.1	52.89
V141	Unburnt	25.3	
V142	Burnt	33.6	32.8
V143	Burnt	40.9	61.7
V071	Unburnt	14.800	
V072	Burnt	17.300	17.12
V073	Burnt	18.800	27.28
F28	Burnt	50.000	31.57
F14	Burnt	42.400	67.5

It can be seen from table that the compressive strength of the cubes that were directly exposed to fire flames increased by 30 -50 %. These results were highly unexpected but on analysis of result some possible reasons have been found.

The compressive strength of cube that were subjected to uniform temperature of 200 °C and 600 °C showed reduction in the compression strength by 13-23 %. The results were upto the expectations as after being damaged to fire concrete always losses some of its strength depending on the range of temperature and duration of temperature to which it has been subjected.

Reason: The compression strength of cubes subjected to direct fire flame increased on account of accelerated curing and internal autoclaving.

Accelerated curing is a method by which high early age strength is achieved in concrete. These techniques are especially useful in the prefabrication industry, wherein high early age strength enables the removal of the formwork within 24 hours, thereby reducing the cycle time, resulting in cost-saving benefits.

A typical curing cycle involves a preheating stage, known as the "delay period" ranging from 2 to 5 hours; heating at the rate of 22 °C/hour or 44 °C/hour until a maximum temperature of 50−82 °C has been achieved; then maintaining at the maximum temperature, and finally the cooling period. The whole cycle should preferably not exceed 18 hours.

At heightened temperatures, the hydration process moves more rapidly and the formation of the Calcium Silicate Hydrate crystals is more rapid. The formation of the gel and colloid is more rapid and the rate of diffusion of the gel is also higher. However, the reaction being more rapid leaves lesser time for the hydration products to arrange suitably, hence the later age strength or the final compressive strength attained is lower in comparison to normally cured concrete.

Internal autoclaving is a phenomenon that may occur in concrete members where, due to heating, moisture is transformed into water vapour. In these conditions chemical and physical changes may take place. This may well activate changes in the microstructure of hydrates and often increases cement paste strength.

The nature of the phase changes will depend upon the mineralogical composition of the cement, its C/S ratio), the amount of fine particles , and the temperature and pressure levels that have been reached. Since the strength of cement paste is increased hence the overall increase in the strength of concrete takes place.

5.1.3 Stress-Strain Behaviour

TABLE 14: Compressive Strength of Concrete Cube

	UNBURNT FIRST MIX SAMPLE 1			
S.No.	LOAD(KN)	STRESS(MPa)	DISPLACEMENT(mm)	STRAIN
1	50	2.222	0	0.000
2	100	4.444	0.1	0.001
3	150	6.667	0.4	0.003
4	200	8.889	0.6	0.004
5	250	11.111	0.8	0.005
6	300	13.333	1.1	0.007
7	350	15.556	1.4	0.009
8	400	17.778	1.9	0.013
9	445.4	19.796	3.2	0.021

Fig. 6a: Stress-Strain Graph of Concrete Cube

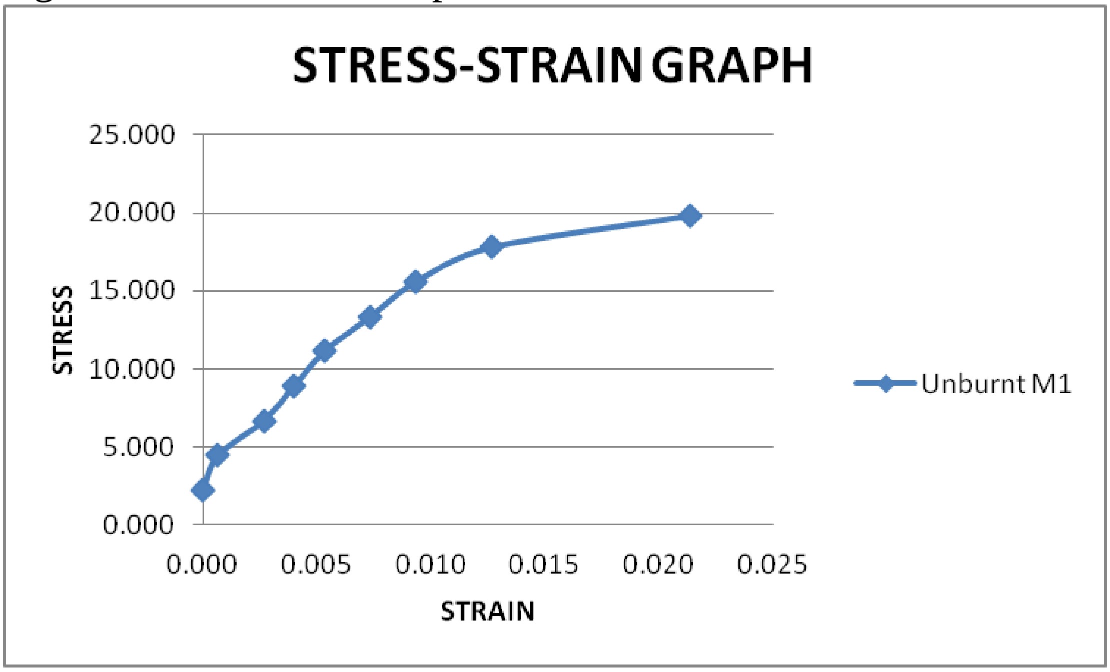


TABLE 15: Compressive Strength of Concrete Cube

	BURNT FIRST MIX SAMPLE 2			
S.No.	LOAD(KN)	STRESS(MPa)	DISPLACEMENT(mm)	STRAIN
1	50	2.222	0.1	0.001
2	75	3.333	0.3	0.002
3	100	4.444	0.4	0.003
4	150	6.667	0.6	0.004
5	200	8.889	0.8	0.005
6	250	11.111	1	0.007
7	300	13.333	1.3	0.009
8	350	15.556	1.6	0.011
9	400	17.778	2	0.013
10	450	20.000	2.6	0.017
11	500	22.222	3.5	0.023
12	550	24.444	4.1	0.027
13	583.3	25.924	4.3	0.029

Fig. 6b: Stress-Strain Graph of Concrete Cube

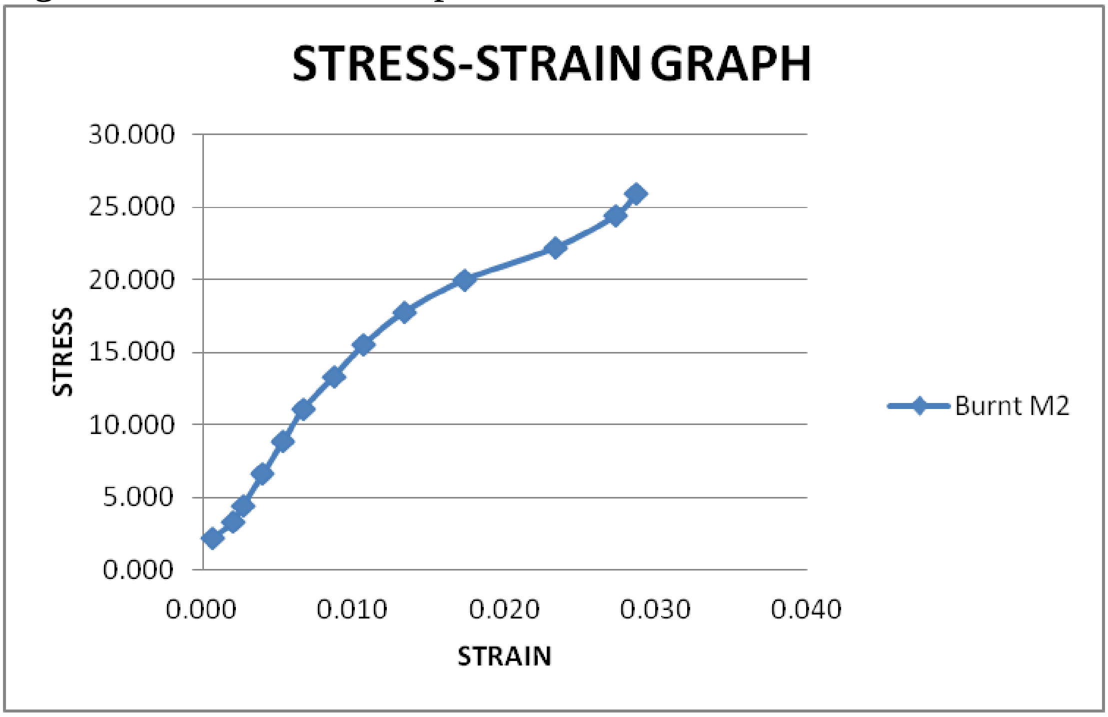


TABLE 16: Compressive Strength of Concrete Cube

	BURNT FIRST MIX SAMPLE 3			
S.No.	LOAD(KN)	STRESS(MPa)	DISPLACEMENT(mm)	STRAIN
1	25	1.111	0.3	0.002
2	50	2.222	0.8	0.005
3	75	3.333	1	0.007
4	100	4.444	1.3	0.009
5	150	6.667	1.7	0.011
6	200	8.889	1.9	0.013
7	250	11.111	2.2	0.015
8	300	13.333	2.4	0.016
9	350	15.556	2.6	0.017
10	400	17.778	2.7	0.018
11	450	20.000	2.9	0.019
12	500	22.222	3	0.020
13	550	24.444	3.2	0.021
14	579.2	25.742	3.8	0.025

Fig. 6c: Stress-Strain Graph of Concrete Cube

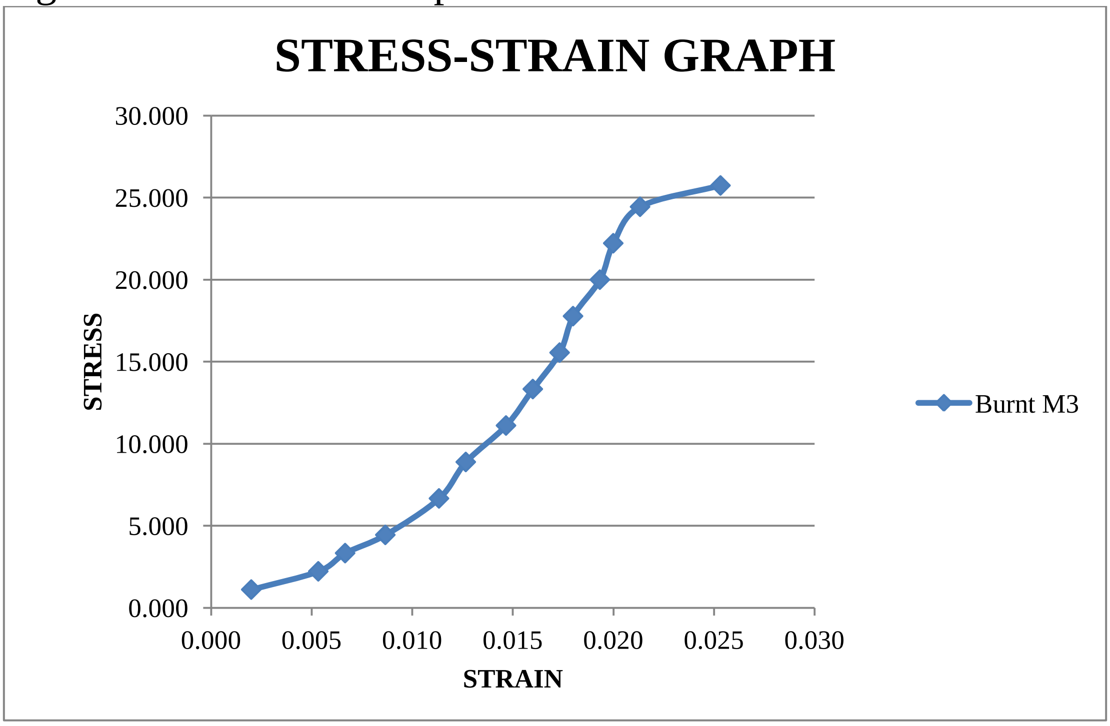


TABLE 17: Compressive Strength of Concrete Cube

	UNBURNT SECOND MIX SAMPLE 1			
S.No.	LOAD(KN)	STRESS(MPa)	DISPLACEMENT(mm)	STRAIN
1	50	2.222	0	0.000
2	100	4.444	0.2	0.001
3	125	5.556	0.4	0.003
4	150	6.667	0.5	0.003
5	200	8.889	0.6	0.004
6	250	11.111	0.7	0.005
7	300	13.333	0.8	0.005
8	325	14.444	0.9	0.006
9	350	15.556	1	0.007
10	400	17.778	1.1	0.007
11	425	18.889	1.2	0.008
12	450	20.000	1.3	0.009
13	475	21.111	1.4	0.009
14	500	22.222	1.5	0.010
15	525	23.333	1.6	0.011
16	550	24.444	1.8	0.012
17	575	25.556	2	0.013
18	586.2	26.053	2.9	0.019

Fig. 6d: Stress-Strain Graph of Concrete Cube

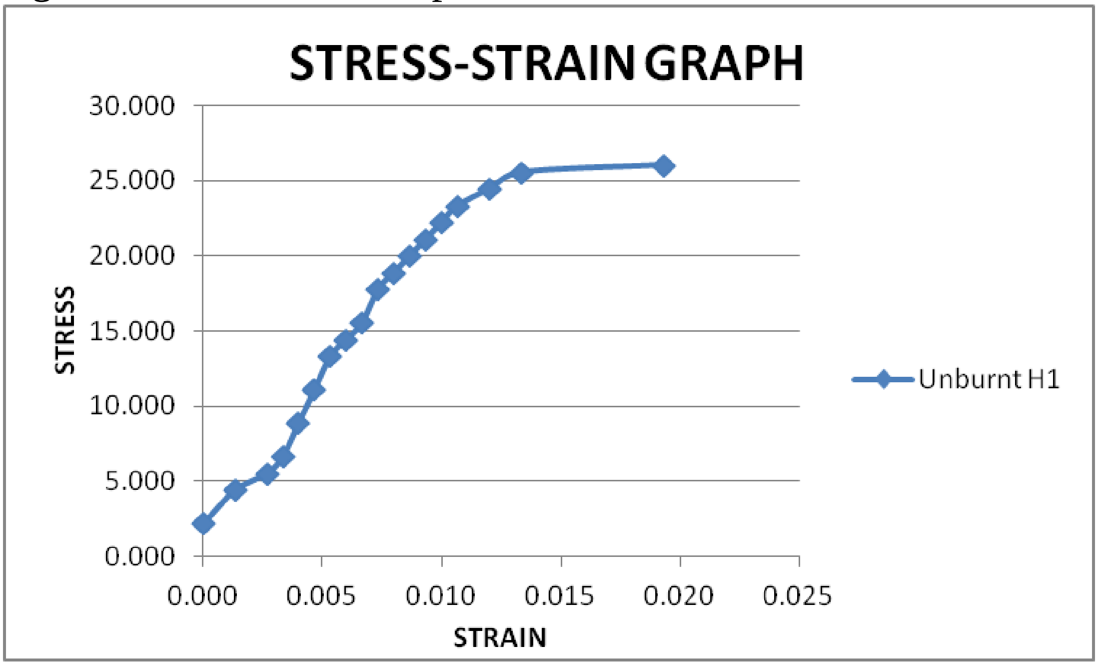


TABLE 18: Compressive Strength of Concrete Cube

	BURNT SECOND MIX SAMPLE 2			
S.No.	LOAD(KN)	STRESS(MPa)	DISPLACEMENT(mm)	STRAIN
1	35	1.556	0.1	0.001
2	100	4.444	0.6	0.004
3	130	5.778	0.7	0.005
4	170	7.556	0.8	0.005
5	200	8.889	1	0.007
6	250	11.111	1.1	0.007
7	300	13.333	1.3	0.009
8	350	15.556	1.4	0.009
9	400	17.778	1.6	0.011
10	450	20.000	1.8	0.012
11	500	22.222	1.9	0.013
12	550	24.444	2.1	0.014
13	600	26.667	2.2	0.015
14	650	28.889	2.4	0.016
15	700	31.111	2.6	0.017
16	750	33.333	2.8	0.019
17	800	35.556	3.1	0.021
18	813.3	36.147	3.5	0.023

Fig. 6e: Stress-Strain Graph of Concrete Cube

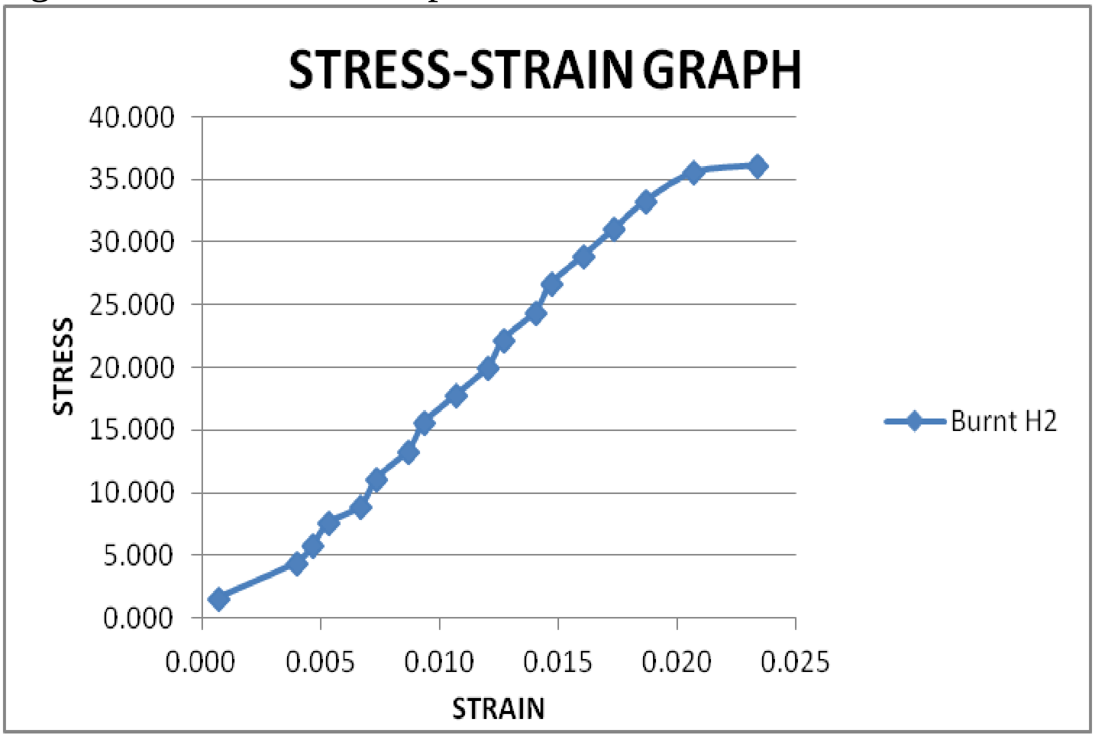


TABLE 19: Compressive Strength of Concrete Cube

	BURNT SECOND MIX SAMPLE 3			
S.No.	**LOAD(KN)**	**STRESS(MPa)**	**DISPLACEMENT(mm)**	**STRAIN**
1	100	4.444	0.9	0.006
2	150	6.667	1.1	0.007
3	200	8.889	1.2	0.008
4	250	11.111	1.4	0.009
5	300	13.333	1.5	0.010
6	350	15.556	1.7	0.011
7	400	17.778	1.8	0.012
8	450	20.000	1.9	0.013
9	500	22.222	2.1	0.014
10	550	24.444	2.2	0.015
11	600	26.667	2.3	0.015
12	650	28.889	2.4	0.016
13	700	31.111	2.5	0.017
14	750	33.333	2.6	0.017
15	800	35.556	2.7	0.018
16	850	37.778	2.8	0.019
17	870	38.667	2.9	0.019
18	882.8	39.236	3.2	0.021

Fig. 6f: Stress-Strain Graph of Concrete Cube

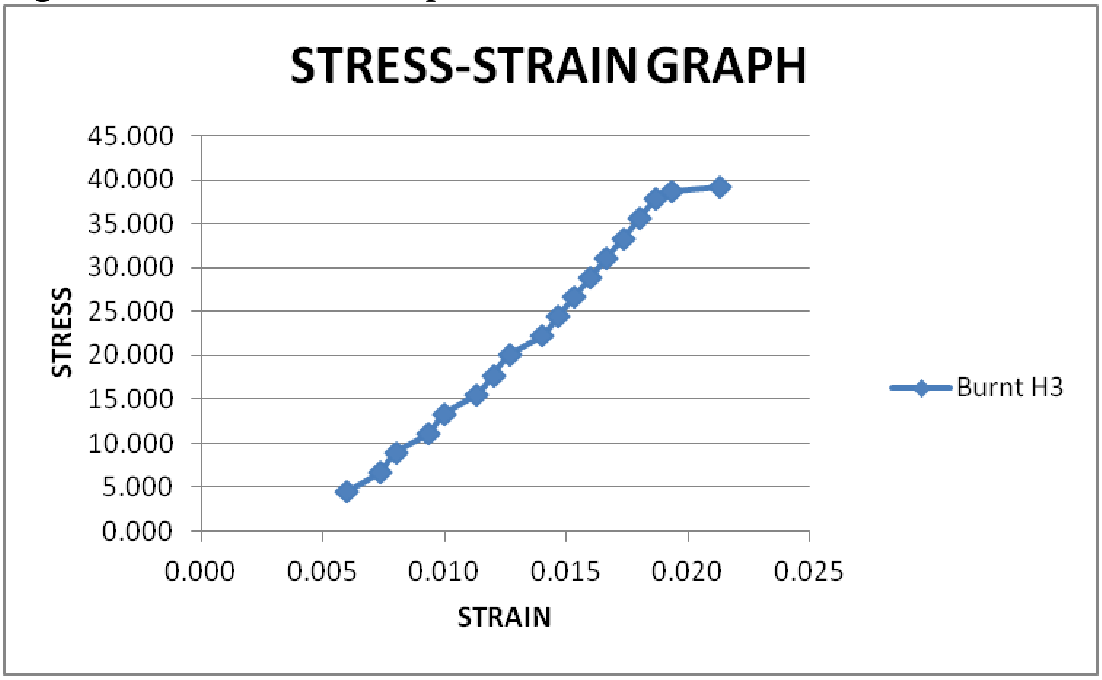


TABLE 20: Compressive Strength of Concrete Cube

	UNBURNT THIRD MIX SAMPLE 1			
S.No.	LOAD(KN)	STRESS(MPa)	DISPLACEMENT(mm)	STRAIN
1	50	5.000	0	0.000
2	100	10.000	0.2	0.002
3	150	15.000	0.5	0.005
4	200	20.000	0.7	0.007
5	250	25.000	1	0.010
6	300	30.000	1.3	0.013
7	332	33.200	2	0.020

Fig. 6g: Stress-Strain Graph of Concrete Cube

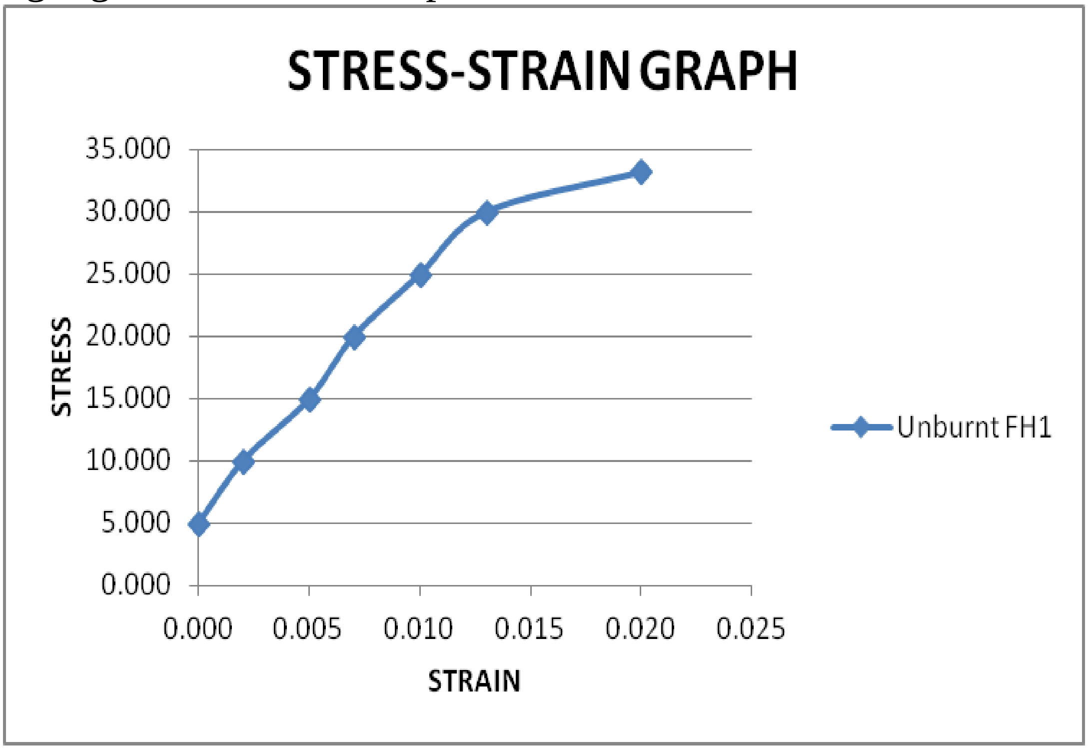


TABLE 21: Compressive Strength of Concrete Cube

	BURNT THIRD MIX SAMPLE 2(200°C)			
S.No.	**LOAD(KN)**	**STRESS(MPa)**	**DISPLACEMENT(mm)**	**STRAIN**
1	50	5.000	0	0.000
2	100	10.000	0.3	0.003
3	125	12.500	0.7	0.007
4	150	15.000	1	0.010
5	200	20.000	1.4	0.014
6	250	25.000	1.9	0.019
7	289	28.900	2.7	0.027

Fig. 6h: Stress-Strain Graph of Concrete Cube

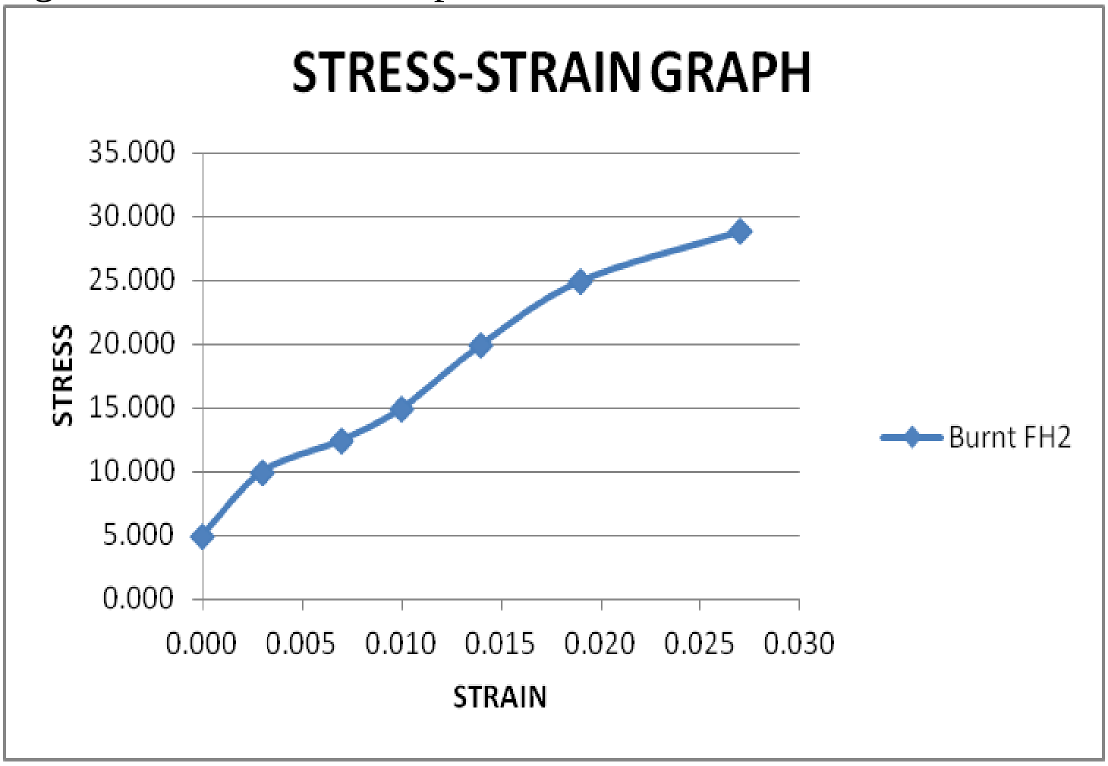
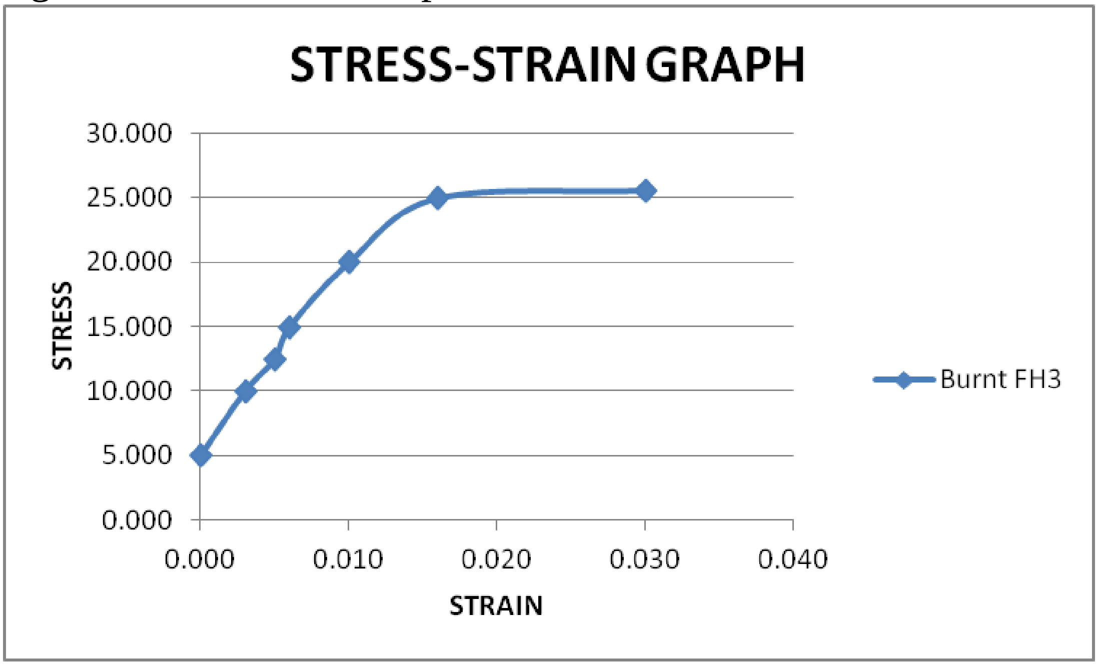


TABLE 22: Compressive Strength of Concrete Cube

	BURNT THIRD MIX SAMPLE 3(600°C)			
S.No.	**LOAD(KN)**	**STRESS(MPa)**	**DISPLACEMENT(mm)**	**STRAIN**
1	50	5.000	0	0.000
2	100	10.000	0.3	0.003
3	125	12.500	0.5	0.005
4	150	15.000	0.6	0.006
5	200	20.000	1	0.010
6	250	25.000	1.6	0.016
7	256	25.600	3	0.030

Fig. 6i: Stress-Strain Graph of Concrete Cube



RETESTING OF PLAIN CONCRETE CUBES

TABLE 23: Compressive Strength of Concrete Cube

	UNBURNT FIRST MIX SAMPLE 1			
S.No.	**LOAD(KN)**	**STRESS(MPa)**	**DISPLACEMENT(mm)**	**STRAIN**
1	50	5.000	0	0.000
2	100	10.000	0.4	0.004
3	150	15.000	0.7	0.007
4	200	20.000	0.9	0.009
5	250	25.000	1.4	0.014
6	300	30.000	1.7	0.017
7	350	35.000	1.9	0.019
8	380	38.000	2.2	0.022

Fig. 6j: Stress-Strain Graph of Concrete Cube

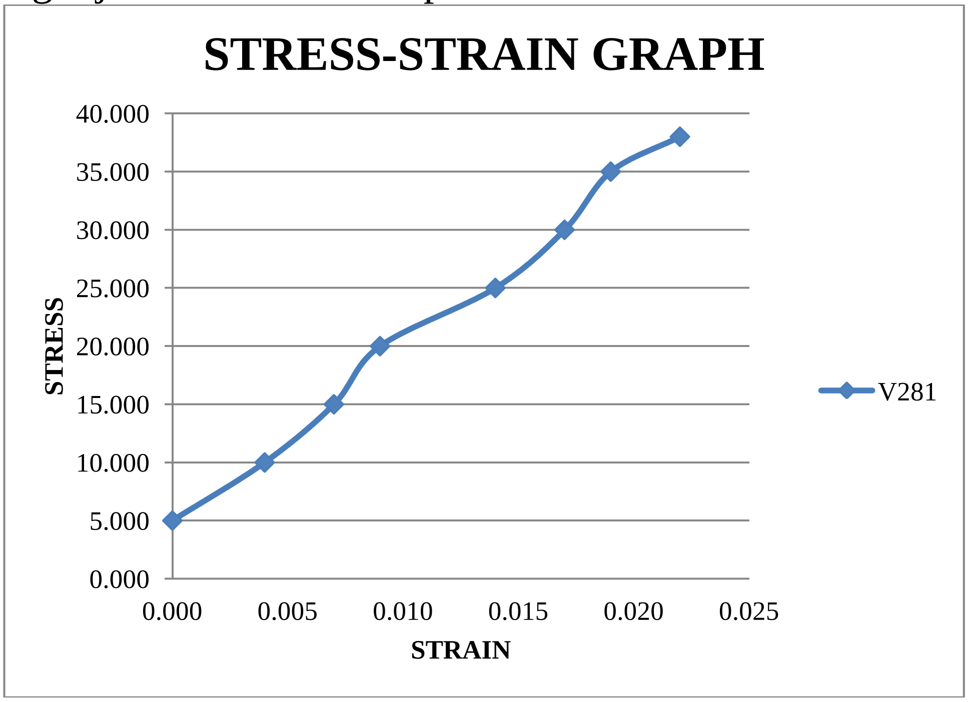


TABLE 24: Compressive Strength of Concrete Cube

	BURNT FIRST MIX SAMPLE 2			
S.No.	**LOAD(KN)**	**STRESS(MPa)**	**DISPLACEMENT(mm)**	**STRAIN**
1	50	5.000	0	0.000
2	100	10.000	0.3	0.003
3	150	15.000	0.5	0.005
4	200	20.000	0.7	0.007
5	250	25.000	1	0.010
6	300	30.000	1.2	0.012
7	350	35.000	1.5	0.015
8	400	40.000	1.8	0.018
9	450	45.000	2.1	0.021
10	500	50.000	2.5	0.025
11	572.9	57.290	2.7	0.027

Fig. 6k: Stress-Strain Graph of Concrete Cube

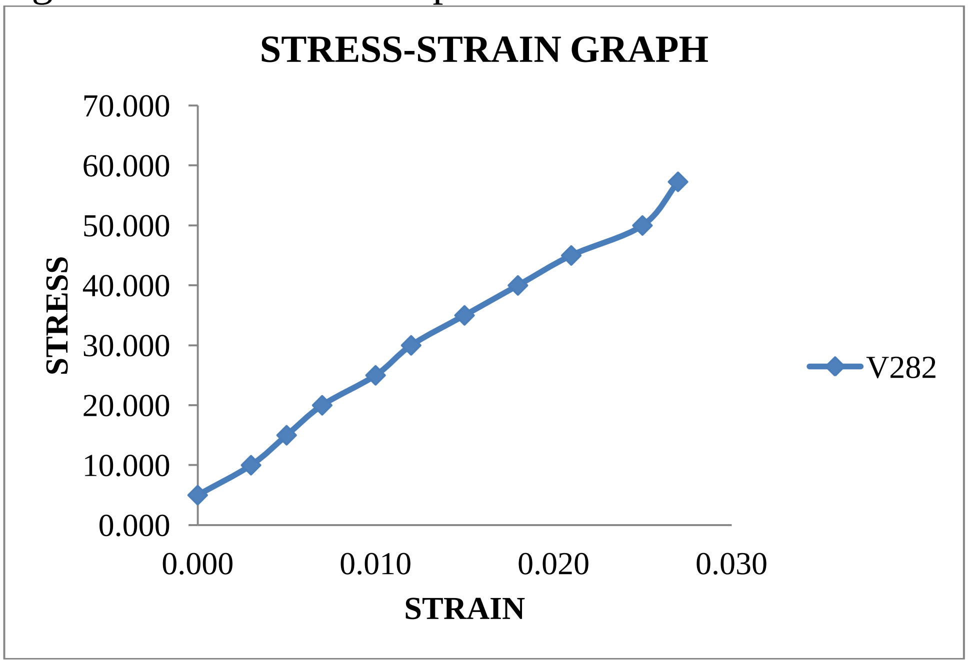


TABLE 25: Compressive Strength of Concrete Cube

	BURNT FIRST MIX SAMPLE 3			
S.No.	LOAD(KN)	STRESS(MPa)	DISPLACEMENT(mm)	STRAIN
1	50	5.000	0	0.000
2	100	10.000	0.1	0.001
3	150	15.000	0.3	0.003
4	200	20.000	0.5	0.005
5	250	25.000	0.7	0.007
6	300	30.000	0.9	0.009
7	350	35.000	1.1	0.011
8	400	40.000	1.4	0.014
9	450	45.000	1.6	0.016
10	500	50.000	1.9	0.019
11	550	55.000	2.3	0.023
12	581	58.100	2.4	0.024

Fig. 6l: Stress-Strain Graph of Concrete Cube

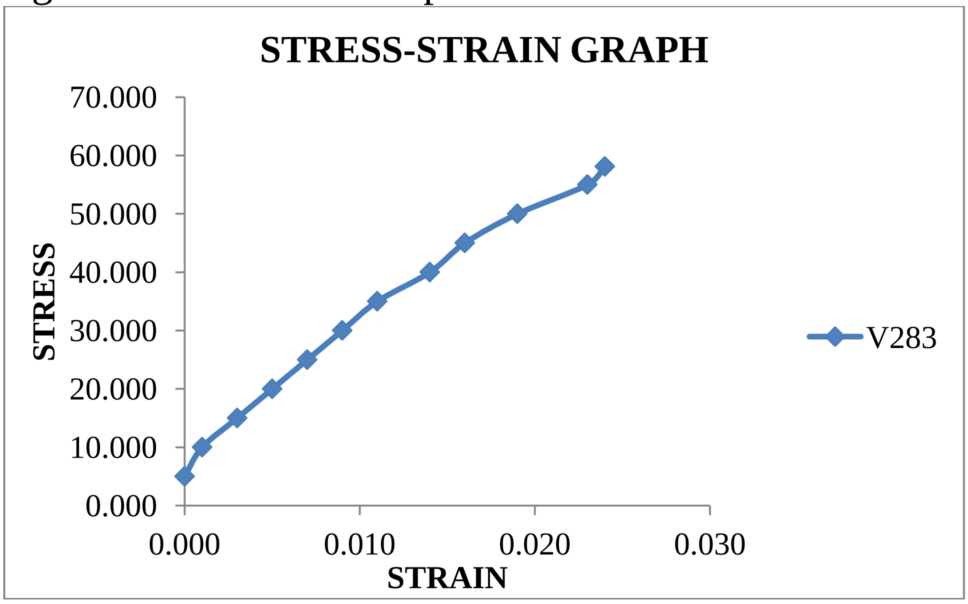


TABLE 26: Compressive Strength of Concrete Cube

	UNBURNT SECOND MIX SAMPLE 1			
S.No.	**LOAD(KN)**	**STRESS(MPa)**	**DISPLACEMENT(mm)**	**STRAIN**
1	50	5.000	0.1	0.001
2	100	10.000	0.5	0.005
3	150	15.000	0.7	0.007
4	200	20.000	0.9	0.009
5	253	25.300	1.1	0.011

Fig. 6m: Stress-Strain Graph of Concrete Cube

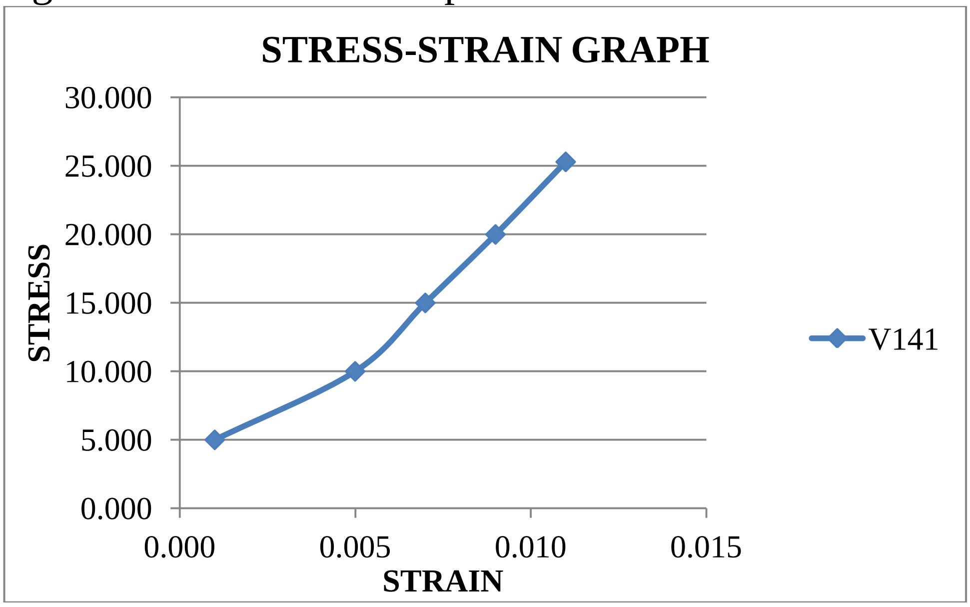


TABLE 27: Compressive Strength of Concrete Cube

	BURNT SECOND MIX SAMPLE 2			
S.No.	LOAD(KN)	STRESS(MPa)	DISPLACEMENT(mm)	STRAIN
1	50	5.000	0.2	0.002
2	100	10.000	0.8	0.008
3	150	15.000	1.1	0.011
4	200	20.000	1.4	0.014
5	250	25.000	1.6	0.016
6	300	30.000	1.9	0.019
7	336	33.600	2.3	0.023

Fig. 6n: Stress-Strain Graph of Concrete Cube

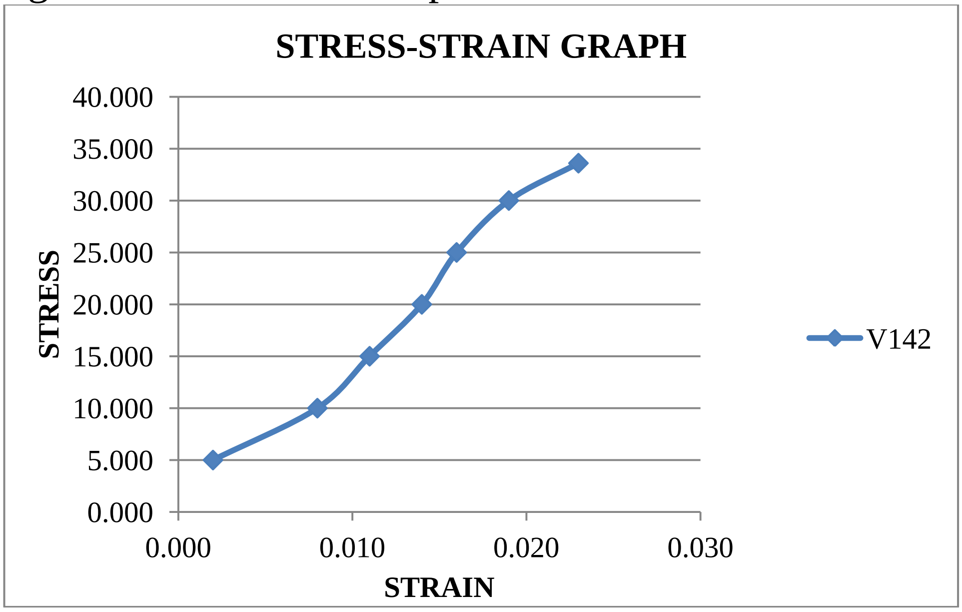


TABLE 28: Compressive Strength of Concrete Cube

	BURNT SECOND MIX SAMPLE 3			
S.No.	**LOAD(KN)**	**STRESS(MPa)**	**DISPLACEMENT(mm)**	**STRAIN**
1	50	5.000	0.1	0.001
2	100	10.000	0.4	0.004
3	150	15.000	0.7	0.007
4	200	20.000	0.9	0.009
5	250	25.000	1.1	0.011
6	300	30.000	1.2	0.012
7	350	35.000	1.4	0.014
8	400	40.000	1.7	0.017
9	409	40.900	1.9	0.019

Fig. 6o: Stress-Strain Graph of Concrete Cube

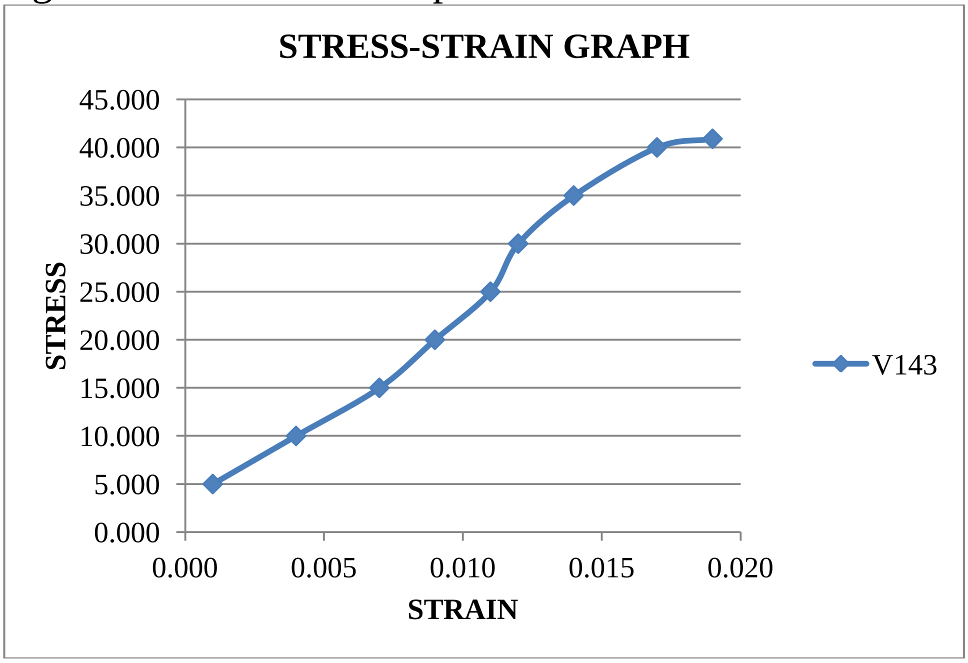


TABLE 29: Compressive Strength of Concrete Cube

	UNBURNT THIRD MIX SAMPLE 1			
S.No.	**LOAD(KN)**	**STRESS(MPa)**	**DISPLACEMENT(mm)**	**STRAIN**
1	50	5.000	0.2	0.002
2	100	10.000	1	0.010
3	147.7	14.770	1.9	0.019

Fig. 6p: Stress-Strain Graph of Concrete Cube

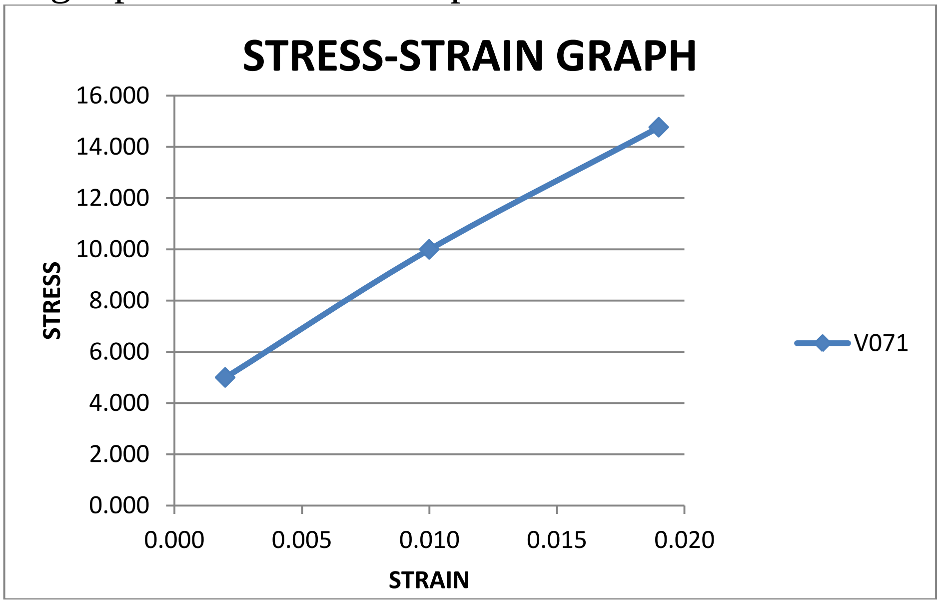


TABLE 30: Compressive Strength of Concrete Cube

	BURNT THIRD MIX SAMPLE 2			
S.No.	**LOAD(KN)**	**STRESS(MPa)**	**DISPLACEMENT(mm)**	**STRAIN**
1	50	5.000	0.1	0.001
2	100	10.000	0.9	0.009
3	173	17.300	1.7	0.017

Fig. 6q: Stress-Strain Graph of Concrete Cube

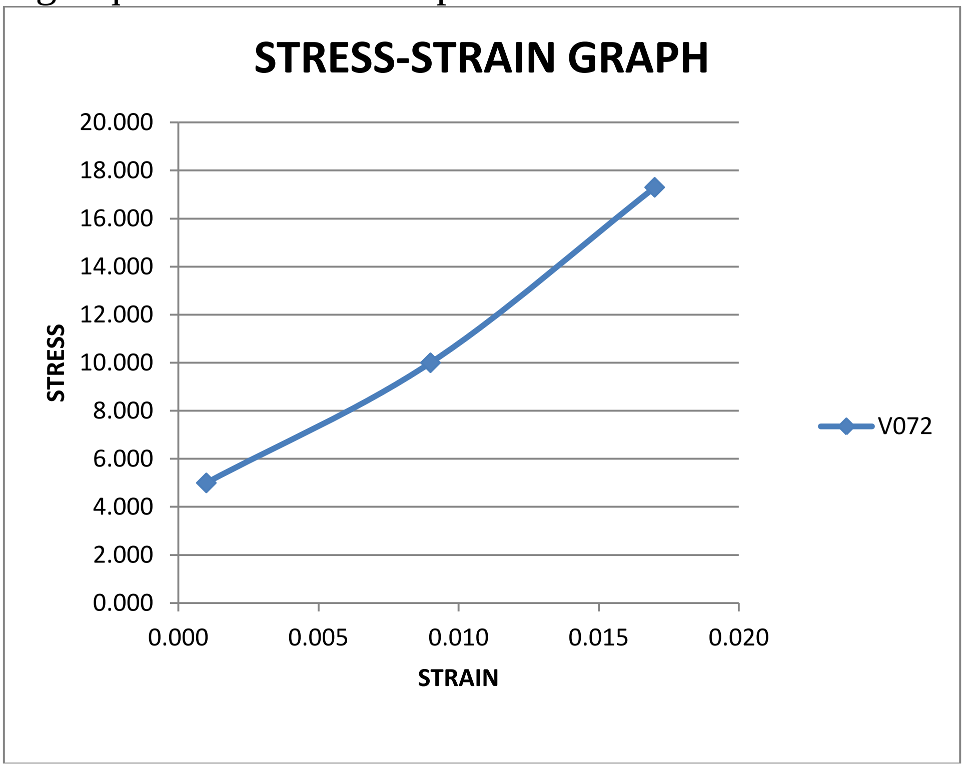


TABLE 31: Compressive Strength of Concrete Cube

	BURNT MIX SAMPLE (28 Days)			
S.No.	**LOAD(KN)**	**STRESS(MPa)**	**DISPLACEMENT(mm)**	**STRAIN**
1	50	5.000	0.1	0.001
2	100	10.000	0.5	0.005
3	150	15.000	0.7	0.007
4	200	20.000	0.9	0.009
5	250	25.000	1.1	0.011
6	300	30.000	1.2	0.012
7	350	35.000	1.3	0.013
8	400	40.000	1.4	0.014
9	450	45.000	1.4	0.014
10	500	50.000	1.5	0.015

Fig. 6r: Stress-Strain Graph of Concrete Cube

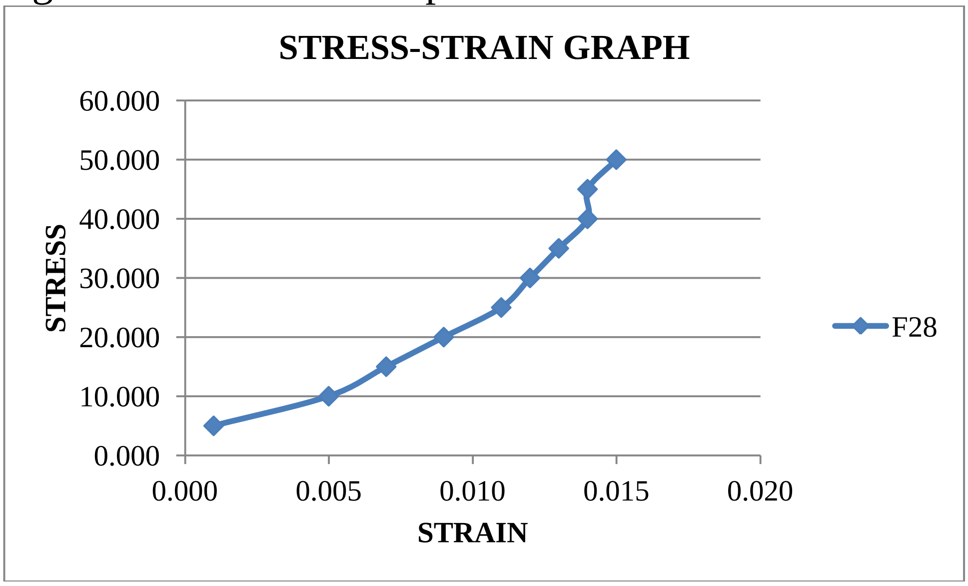
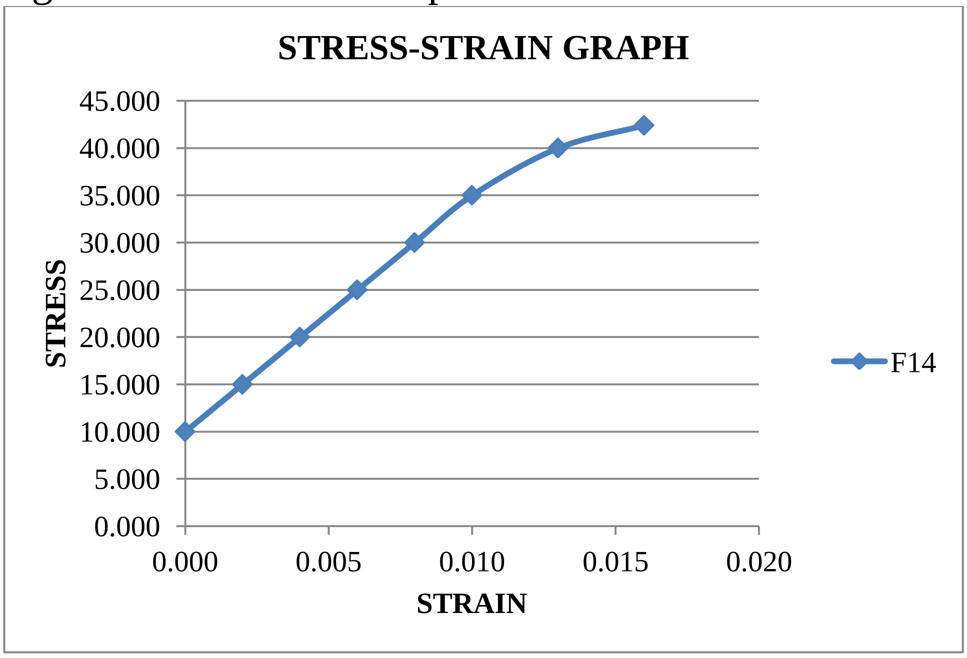


TABLE 32: Compressive Strength of Concrete Cube

	BURNT MIX SAMPLE (14 DAYS)			
S.No.	**LOAD(KN)**	**STRESS(MPa)**	**DISPLACEMENT(mm)**	**STRAIN**
1	100	10.000	0	0.000
2	150	15.000	0.2	0.002
3	200	20.000	0.4	0.004
4	250	25.000	0.6	0.006
5	300	30.000	0.8	0.008
6	350	35.000	1	0.010
7	400	40.000	1.3	0.013
8	424	42.400	1.6	0.016

Fig. 6s: Stress-Strain Graph of Concrete Cube



5.1.4 Analysis of Stress Strain Behaviour

A typical relationship between stress and strain for normal strength concrete has been shown above. After an initial linear portion lasting up to about 30 – 40% of the ultimate load, the curve becomes non-linear, with large strains being registered for small increments of stress. The non-linearity is primarily a function of the coalescence of microcracks at the paste-aggregate interface.

The ultimate stress is reached when a large crack network is formed within the concrete, consisting of the coalesced microcracks and the cracks in the cement paste matrix. The strain corresponding to ultimate stress is usually around 0.003 for normal strength concrete. The stress-strain behaviour in tension is similar to that in compression.

From the stress-strain graph that has been shown above it is clear that the all the unburnt sample have shown around similar behaviour on application of load. Initially for all the samples the stress-strain curve is linear and after that it becomes no-linear. The linear portion of curve denotes that stage due to which microcracks starts to build in the concrete due to application of external load while non-linear portion of graph is present due to interconnection of already developed microcracks.

While the stress –strain curve for burnt and heated samples has shown a large variation in their behaviour. Some cube had shown linear behaviour while others have not. The shape of the curve for some of the cubes was concave upwards while some of the cubes have shown convex downwards. The concave upward curves shows that those cubes has undergone more rigorous burning than other cubes because of which too much thermal and shrinkage cracks got developed in the concrete in heating and that is even on low level of stress it has undergone large strain change.

5.2 Reinforced Concrete beam

5.2.1 Visual Observation

1) After direct exposure to fire flame the concrete surface turned black from grey.

   Reason: Colour change from grey to black was on account of external factors. It occurred due to incomplete combustion of wood that was used for burning.

$$2C + O_2 \rightarrow 2CO \text{ (Carbon Monoxide)}$$

Before burning

After burning

Fig 7: Pictures Showing Change in Surface Colour

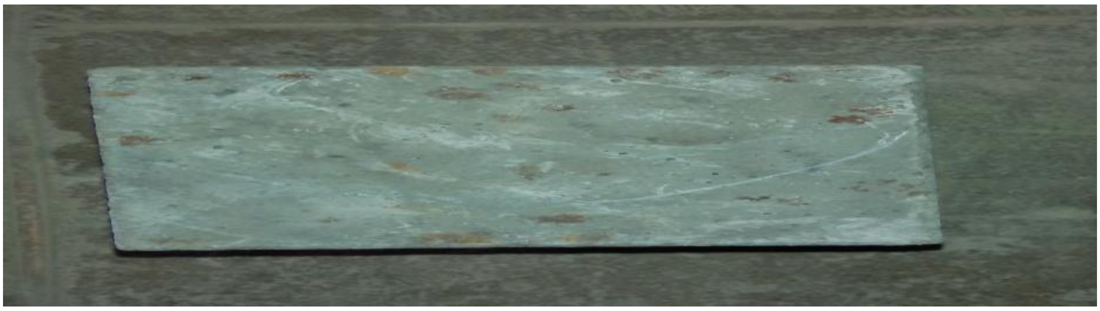
2) The concrete after burning at 600 °C turned reddish from grey colour.

Reasons: The colour change of concrete after burning from grey to reddish is due to presence of siliceous aggregate in the concrete . It is generally agreed that when heated to between 300 ◦C and 600 ◦C concrete containing siliceous aggregates will turn red; between 600◦C and 900◦C, whitish-grey; and between 900◦C and 1000◦C, a buff colour is present.

The colour change of heated concrete results principally from the gradual water removal and dehydration of the cement paste, but also transformations occurring within the aggregate. The most intense colour change, the appearance of red colouration, is observed for siliceous riverbed aggregates containing iron. This colouration is caused by the oxidation of mineral components.

While siliceous aggregates turn red when heated, the aggregates containing calcium carbonate get whitish. Due to calcination process $CaCO3$ turns to lime and give pale shades of white and grey.

Before burning



After burning

Fig 8: Pictures showing change in colour of concrete

3) After being exposed to direct fire flames and uniform temperature of 600 °C minor cracks developed on the surface that were visible from naked eyes only. The pattern of cracked developed is web pattern and vertical cracks.

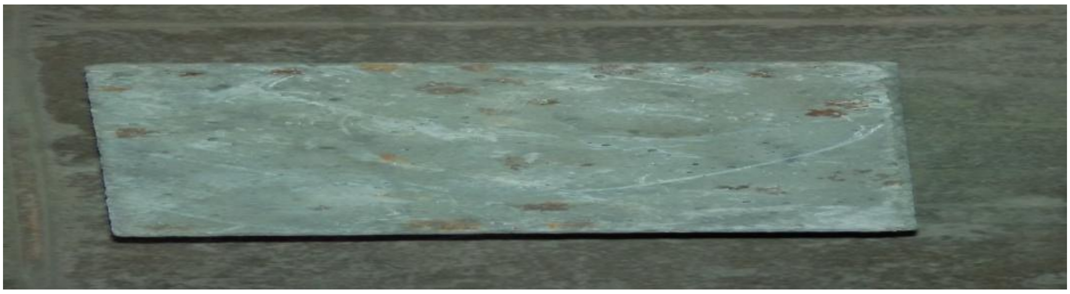
Reason: Thermal cracking occurred in the samples. It occurs due to excessive temperature difference within a concrete structure or its surroundings. This difference causes the cooler portion to contract more than the warmer portion, which restrain the contraction. Random cracks develop on surfaces. This restraint creates tensile stresses that crack the concrete as a result of this uncontrolled temperature difference across the cross-section.

There has been the loss of water while burning. It may have induced shrinkage as due to combined effect of thermal difference and shrinkage, vertical cracks from base are usually observed.

In reinforced concrete due to presence of reinforcement there had been disproportionate thermal expansion of steel and concrete. Generally on application of high temperature steel reinforcement expands more than the concrete and because of this difference in the expansion of the two materials the large no of cracks, deep and wider appeared on the surface of the beam.

There are two types of cracks that occurred. First type of cracks are arising due to thermal expansion. They are deep, wide and run along the length of beam, it is due to differential expansion of steel and concrete. some thermal cracks are minor and randomly oriented depicting a pattern of web. these type of cracks occur due to difference in expansion of aggregates and cement matrix. On exposure to temperatures aggregates and cement matrix which mainly contains product of hydration expands differently. Aggregates show greater expansion than the cement matrix. Second types of cracks that occur are shrinkage cracks. Such cracks start from edges and run perpendicular to edges.

Before burning



After burning

Fig 9: Pictures showing surface cracks

5.2.2 Flexure Strength Test

Flexure strength test was performed on both unburnt and burnt beam that was damaged by direct flame exposure. Three point loading was applied by universal testing machine.

The flexure strength of beams was recorded and is presented in Table shown below.

TABLE 33: Flexure Strength of Beam

NAME OF SAMPLE	CONDITION	ULTIMATE LOAD(KN)	FLEXURE STRENGTH(MPa)
B1	UNBURNT	92.5	36.075
B2	BURNT	64.75	25.25

The flexure strength of reinforced concrete beam was affected adversely by fire flame. The residual flexure strength was 25.25 MPa.

It can be indicated from the results in table that the ultimate load capacity of the beams is adversely influenced by the fire flame exposure and this deleterious effect decreases the ultimate load capacity by 31.5%.

Reason

Flexure strength loss of heated concretes results mainly from the change that occurs in the concrete microstructure during the heating process. Some complicated processes of shrinkage, decomposition, expansion and crystal destruction occur during fire.

Moreover the primary factor that acts most in reducing the the ultimate load carrying capacity of the reinforced beam is disproportionate thermal expansion of steel and concrete and aggregates and cement matrix. In reinforced concrete due to presence of reinforcement there had been disproportionate thermal expansion of steel and concrete.

Generally on application of high temperature steel reinforcement expands more than the concrete and because of this difference in the expansion of the two materials the large no of cracks deep and wider appeared on the surface of the beam.

There many minor and deep cracks formed in reinforced concrete beam after burning whose presence reduces the ultimate load carrying capacity of the reinforced beam and it fails early.

5.2.3 Load Deflection Behaviour

Table 34: load and deflection of beams

UNBURNT BEAM	
LOAD(KN)	**DEFLECTION(mm)**
0	0
10	0.9
20	1.8
30	4.6
40	8.9
50	10.1
60	13.7
70	16.8
80	19.5
90	22.4
92.5	24.5
BURNT BEAM	
LOAD(KN)	**DEFLECTION(mm)**
0	0
10	1.0
20	2.1
30	4.9
40	9.2
50	13.5
60	17.6
64.75	19.5

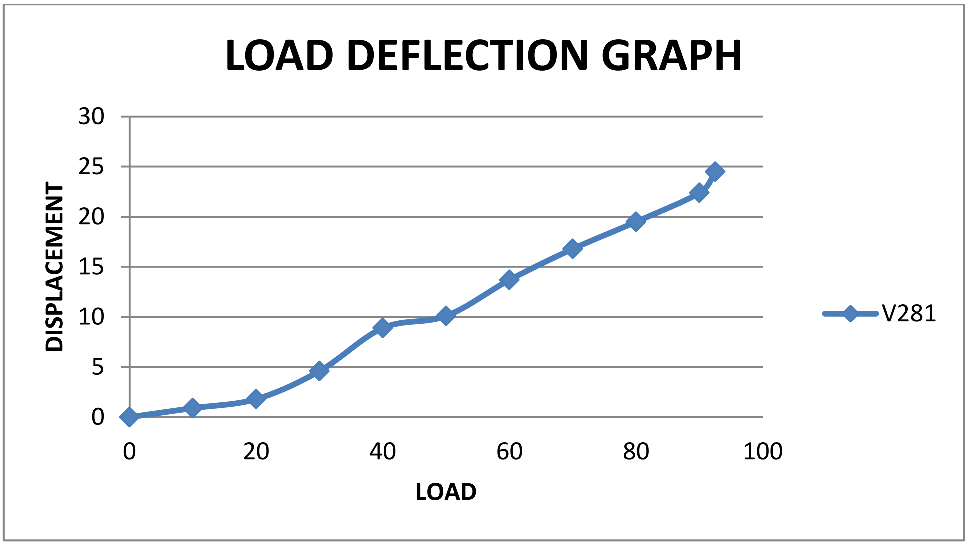


Fig 10a: Load Deflection Behaviour of Unburnt Beam

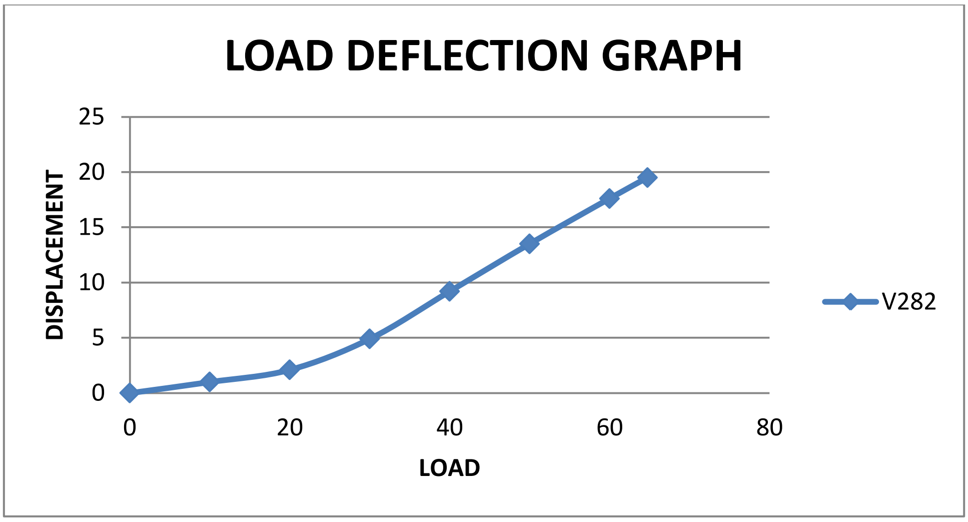


Fig 10b: Load Deflection Behaviour of Burnt Beam

5.2.4 Load Deflection Analysis

A typical relationship between stress and strain for normal strength concrete has been shown above. After an initial linear portion lasting up to about 30 – 40% of the ultimate load, the curve becomes non-linear, with large strains being registered for small increments of stress. The non-linearity is primarily a function of the coalescence of microcracks at the paste-aggregate interface.

The ultimate stress is reached when a large crack network is formed within the concrete, consisting of the coalesced microcracks and the cracks in the cement paste matrix.. Initially for all the samples the stress-strain curve is linear and after that it becomes non-linear. The linear portion of curve denotes that stage due to which microcracks starts to build in the concrete due to application of external load while non-linear portion of graph is present due to interconnection of already developed microcracks.

If we compare the load deflection graph of burnt and unburnt beam we will notice that at later stages the graph for burnt beam is steeper than the graph for unburnt beam. This shows the reduction in the stiffness of the burnt beam.

The maximum deflection at ultimate load increases by about 35% which shows clearly reduced stiffness behaviour. At 64.75 KN the deflection is around 15.2 mm in the unburnt beam while it is around 20.1 in burnt beam. This reduction in the stiffness of the reinforced beam specimens subjected to fire flame can be attributed to the following two parameters; the first is the reduction in the moment of inertia of the section due to cracking initiating due to the effect of fire, whereas, the second is the reduction in the modulus of elasticity (E) of concrete and steel.

CONCLUSION

If the hydration is not completed the plain concrete will undergo accelerated curing and internal autoclaving when exposed to high temperature. If the hydration is still not complete and plain concrete is exposed to high temperature its strength will reduce. Exposure to uniform temperature of 600^{0}C for a duration of three hours results in cracking and color change of plain concrete. There is no spalling in M30 plain concrete.

The compression strength of cubes subjected to direct fire flame increased on account of accelerated curing and internal autoclaving since the hydration was still not completed and there was enough moisture present in the concrete specimen.

Accelerated curing helped in achieving the remaining strength quickly as due to heightened temperature the hydration reactions are speeded up while due to internal autoclaving which occurs due to conversion of internal water into vapours, thus a pressure is created and due to combined effect of pressure and temperature the strength of cement paste is enhanced. Hence overall strength of concrete is increased.

When the hydration reaction is almost complete and enough moisture is not present in the concrete then subjecting concrete to high temperatures leads to reduction in strength because the increase in temperature results in water evaporation, C-S-H gel dehydration, calcium hydroxide and calcium aluminates decomposition etc. Along with the increase in temperature, changes in the aggregate take place. Due to those changes, concrete strength and modulus of elasticity gradually decreases.

The ultimate load capacity of the beam has decreased by 30%. during exposure to fire there occurs disproportionate expansion of the steel and concrete and aggregates and cement matrix. This results in minor and deep cracks in concrete and as a result the load carrying capacity of beam decreases.

The stiffness of the beam also reduces by about 35% as reinforced beam after burning shows greater deflection on the application of the external load from the universal testing machine. This reduction in the stiffness of the reinforced beam specimens subjected to fire flame can be attributed to the following two parameters; the first is the reduction in the moment of inertia of the section (section modulus) I_{eff} due to cracking initiating due to the effect of fire, whereas, the second is the reduction in the modulus of elasticity (E) of concrete and steel.

APPENDIX A

TABLE 1: Grading of Fine Aggregates from IS 383-1970

IS Sieve Designation	Percentage Passing for Grading Zone I	Grading Zone II	Grading Zone III	Grading Zone IV
10 mm	100	100	100	100
4·75 mm	90-100	90-100	90-100	95-100
2·36 mm	60-95	75-100	85-100	95-100
1·18 mm	30-70	55-90	75-100	90-100
600 micron	15-34	35-59	60-79	80-100
300 micron	5-20	8-30	12-40	15-50
150 micron	0-10	0-10	0-10	0-15

TABLE 2: Experimental Values of Fine Aggregates

GRADING OF FINE AGGREGATES				
SIEVE SIZE	WEIGHT RETAINED Gm	CUMULATIVE WT. RETAINED gm	WEIGHT PASSED gm	% WT. PASSED gm
4.75mm	73.7	73.7	926.3	92.63
2.00mm	130.2	203.9	796.1	79.61
1.00mm	105.4	309.3	690.7	69.07
600µ	96	405.3	594.7	59.47
425µ	107.6	512.9	487.1	48.71
212µ	355.4	868.3	131.7	13.17
150µ	43.7	912	88	8.8
PAN	87.9	999.9	0.1	0.01

TABLE 3: Grade of Concrete

Group	Grade Designation	Specified Characteristic Compressive Strength of 150 mm Cube at 28 Days in N/mm^2
(1)	(2)	(3)
Ordinary Concrete	M 10	10
	M 15	15
	M 20	20
Standard Concrete	M 25	25
	M 30	30
	M 35	35
	M 40	40
	M 45	45
	M 50	50
	M 55	55
High Strength Concrete	M 60	60
	M 65	65
	M 70	70
	M 75	75
	M 80	80

TABLE 4: Environmental Exposure Conditions

Sl No.	Environment	Exposure Conditions
(1)	(2)	(3)
i)	Mild	Concrete surfaces protected against weather or aggressive conditions, except those situated in coastal area.
ii)	Moderate	Concrete surfaces sheltered from severe rain or freezing whilst wet
		Concrete exposed to condensation and rain
		Concrete continuously under water
		Concrete in contact or buried under non-aggressive soil/ground water
		Concrete surfaces sheltered from saturated salt air in coastal area
iii)	Severe	Concrete surfaces exposed to severe rain, alternate wetting and drying or occasional freezing whilst wet or severe condensation.
		Concrete completely immersed in sea water
		Concrete exposed to coastal environment
iv)	Very severe	Concrete surfaces exposed to sea water spray, corrosive fumes or severe freezing conditions whilst wet
		Concrete in contact with or buried under aggressive sub-soil/ground water
v)	Extreme	Surface of members in tidal zone
		Members in direct contact with liquid/ solid aggressive chemicals

TABLE 5: Minimum Cement Content, Maximum Free Water-Cement Ratio and Minimum Grade of Concrete for Different Exposure with Normal Weight Aggregate of 20mm Nominal Maximum Size

Sl No.	Exposure	Plain Concrete		
		Minimum Cement Content kg/m³	Maximum Free Water-Cement Ratio	Minimum Grade of Concrete
1)	(2)	(3)	(4)	(5)
i)	Mild	220	0.60	–
iii)	Moderate	240	0.60	M 15
iii)	Severe	250	0.50	M 20
iv)	Very severe	260	0.45	M 20
v)	Extreme	280	0.40	M 25

TABLE 6: Adjustment to Minimum Cement Contents for Aggregates other than 20 mm Nominal Maximum Size

Sl No.	Nominal Maximum Aggregate Size mm	Adjustments to Minimum Cement Contents in Table 5 kg/m³
(1)	(2)	(3)
i)	10	+40
ii)	20	0
iii)	40	–30

TABLE 8: Assumed Standard Deviation

Grade of Concrete	Assumed Standard Deviation N/mm²
M 10, M 15	3.5
M 20, M 25	4.0
M 30, M 35, M 40, M 45, M 50	5.0

TABLE 9: Values of t

Accepted Proportion of Low Results	t
1 in 5	0·84
1 in 10	1·28
1 in 15	1·50
1 in 20	1·65
1 in 40	1·86
1 in 100	2·33

TABLE 10: Appropriate Air Content

Nominal Maximum Size of Aggregate mm	Entrapped Air, as Percentage of Volume of Concrete
10	3·0
20	2·0
40	1·0

TABLE 11: Approximate Sand and Water Contents per Cubic Metres of Concrete for Grades Upto M35

NOMINAL MAXIMUM SIZE OF AGGREGATE mm	WATER CONTENT*, PER CUBIC METRE OF CONCRETE kg	SAND AS PERCENT OF TOTAL AGGREGATE BY ABSOLUTE VOLUME
10	208	40
20	186	35
40	165	30

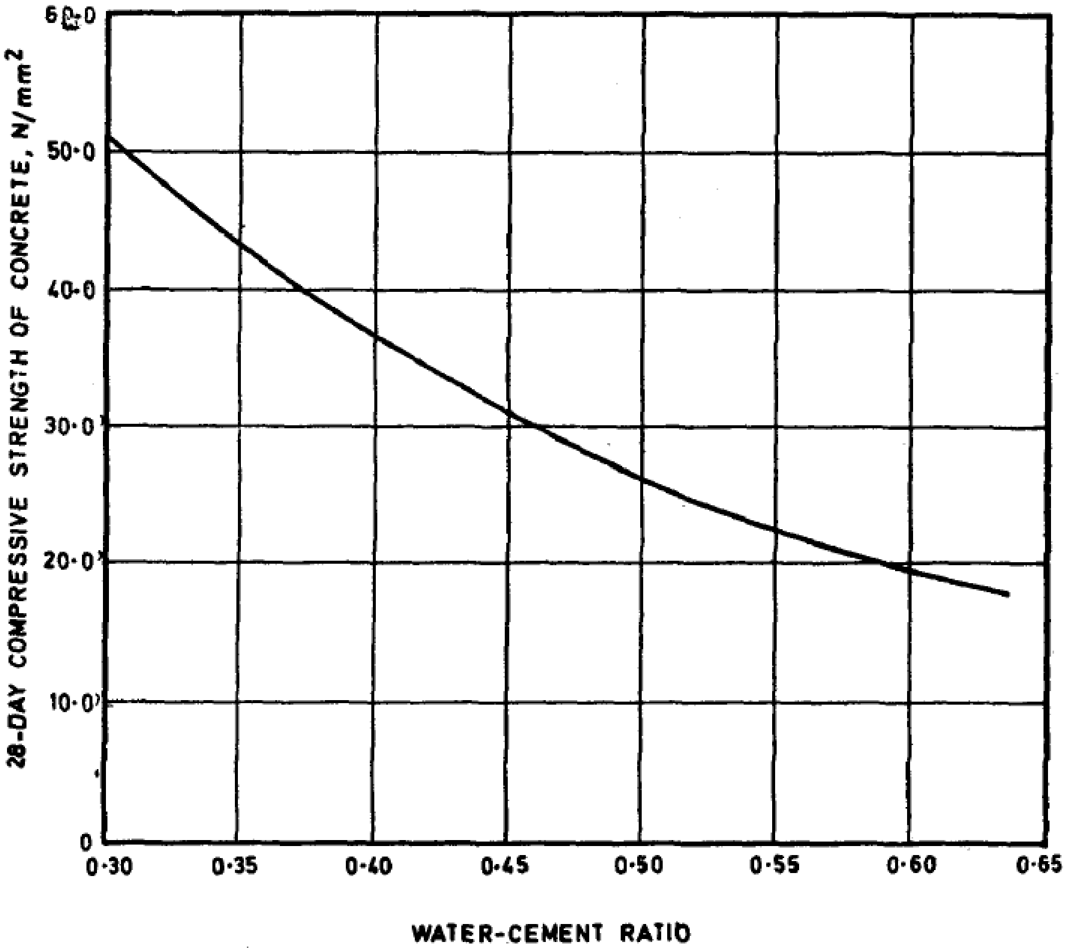
*Water content corresponding to saturated surface dry aggregate.

TABLE 12: Adjustment of Values in Water Content and Sand Percentage for Other Conditions

CHANGE IN CONDITION STIPULATED FOR TABLES 4 AND 5	ADJUSTMENT REQUIRED IN	
	Water Content	Percent, Sand in Total Aggregate
(1)	(2)	(3)
For sand conforming to grading Zone I, Zone III or Zone IV of Table 4 of IS : 383-1970*	0	+ 1·5 percent for Zone I − 1·5 percent for Zone III − 3·0 percent for Zone IV
Increase or decrease in the value of compacting factor by 0·1	± 3 percent	0
Each 0·05 increase or decrease in free water-cement ratio	0	± 1 percent
For rounded aggregate	− 15 kg/m^3	− 7 percent

APPENDIX B

Fig. 1: Graph between 28 day Compressive Strength and Water/Cement Ratio



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