Study on Exposure of High Temperature on Performance of Concrete

By

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14MCLC02

DEPARTMENT OF CIVIL ENGINEERING

INSTITUTE OF TECHNOLOGY

NIRMA UNIVERSITY

AHMEDABAD-382481

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Study on Exposure of High Temperature on Performance of Concrete

Major Project

Submitted in Partial Fulfillment of the Requirements

For the degree of

Master of Technology in Civil Engineering

(Computer Aided Structural Analysis & Design)

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Declaration

This is to certify that

a. The thesis comprises my original work towards the Degree of Master of Technology in Civil Engineering (Computer Aided Structural Analysis and Design) at Nirma University and has not been submitted elsewhere for a degree.

b. Due acknowledgment has been made in the text to all other material used.

Harsh M. Bhadja

Certificate

This is to certify that Major Project entitled **"Study on Exposure of High Temperature on Performance of Concrete"** submitted by **Mr. Harsh M. Bhadja (14MCLC02)**, towards the partial fulfillment of the requirements for the degree of Master of Technology in Civil Engineering (Computer Aided Structural Analysis and Design) of Nirma University, Ahmedabad is the record of work carried out by him under my supervision and guidance. In my opinion, the submitted work has reached a level required for being accepted for examination. The results embodied in this major project, to the best of my knowledge, haven't been submitted to any other university or institution for award of any degree or diploma.
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Abstract

Concrete is a heterogeneous material with relatively inert aggregate that is held together by hydrated Portland cement paste. When concrete is exposed to high temperatures, changes in mechanical & physical properties occur. Concrete at elevated temperature is sensitive to the temperature level, heating rate, thermal cycling and temperature duration. Changes in mechanical & physical properties may result in undesirable structural failure. Therefore the properties of concrete retained after a fire are of still importance for determining the load carrying capacity and for reinstating fire-damaged constructions procedure. Experimental investigation is conducted on four different types of concrete mixes namely M25(PCC), M25(FRC), M60(PCC) & M60(FRC) exposed to different elevated temperatures namely 300°C, 500°C, 700°C & 900°C, respectively. Focus of this study is to evaluate mechanical properties & physical properties of concrete mixes exposed to different elevated temperatures. Mechanical properties taken into considerations are compressive strength, split tensile strength, flexural strength, Modulus of Elasticity & bond strength etc. Physical properties incorporated in this study are spalling effect, Cracking, weight loss etc. Hooked steel fibres have been incorporated in fibre reinforced concrete mixes namely M25(FRC) & M60(FRC). Plain concrete specimens cast from different mixes have been water cured for duration of 28 days. After curing, specimens have been exposed to different elevated temperatures at 300°C, 500°C, 700°C & 900°C, respectively. Specimens have been exposed to different temperature ranges for duration of 1 hour at target temperature as per the experimental time-temperature curve. Experimental time-temperature curve has been derived from IS:3809(1979) standard codal time-temperature curve. After temperature exposure, specimens have been allowed to cool for 24 hours duration at room temperature till steady state condition is achieved. After completion of cooling period, destructive testing has been carried out.

Experimental investigation demonstrates that there is a minor damaging effect in terms of spalling for M25(PCC) & M25(FRC) at 300°C & 500°C, respectively. Damaging effect in terms of spalling is found to be higher in M25(FRC) as compared to M25(PCC) exposed at 700°C & 900°C, respectively. This might be due to the expansion of steel fibres which causes debonding of concrete from steel fibres. From investigation, it is found that M60(PCC) and M60(PCC) are prone to damage in terms of spalling even at 300°C & 500°C, respectively. Weight loss & Crack initiation in M60(PCC) mix is found to be higher as compared to M60(FRC) mix for each temperature range, respectively. Visual observation shows that M60(PCC) & M60(FRC) mixes are more vulnerable to damage as compared to M25(PCC) & M25(FRC) exposed to each temperature range.

Experimental inspection shows that incorporation of steel fibres in M25(FRC) & M60(FRC) in unheated condition are found to be enhancing mechanical properties. Percentage loss in mechanical properties for M25(PCC) & M60(PCC) are higher for each temperature range, respectively. Percentage loss in mechanical properties for M25(FRC) & M60(FRC) are lower as compared to M25(PCC) & M60(PCC) mixes for each temperature range, respectively. Steel fibre incorporation is found to be mitigating strength loss in mechanical properties, such as compressive strength, split tensile strength, flexural strength & modulus of elasticity, for M25(FRC) & M60(FRC) for each temperature range, respectively. It has been found from test result that there is no any beneficial effect of steel fibre on residual bond strength for M25(FRC) & M60(FRC) for each temperature range, respectively.

Destructive failure pattern of M25(PCC) & M60(PCC) mixes are found to be sudden while for M25(FRC) & M60(FRC) it is gradual, which is due to steel fibre incorporation.

Additionally, behaviour of RC columns have been studied exposed to 900°C. RC columns from 4 mixes have been cast having dimension of 150×150×1000 mm with equal amount of reinforcement. In each mix of columns, average result of two columns have been considered as final result. Mechanical properties of RC columns such as ultimate failure load, deflection, stress-strain, failure modes & cracking patterns have been included in this study. Test results demonstrate that in unheated condition, steel fibre incorporation in RC columns[M25(FRC) & M60(FRC)] enhances ultimate load carrying capacity in minor amount. Incorporation of steel fibre enhances displacement ductility for M25(FRC) & M60(FRC) columns in unheated and heated conditions, respectively. Plain concrete columns[M25(PCC) & M60(PCC)] showed sudden failure while that of fibre reinforced columns[M25(FRC) & M60(FRC)] showed gradual failure after exposed to 900°C. Fibre reinforced columns having grade M25(FRC) & M60(FRC) undergo large vertical and axial deformation before failure. Incorporation of steel fibre in M25(FRC) & M60(FRC) is found to reducing crack development and arresting the crack propagation.

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I would first of all like to thank **Dr. U. V. Dave**, Guide M.Tech. Department of Civil Engineering (CASAD), Institute of Technology, Nirma University, Ahmedabad whose keen interest and excellent knowledge base helped me to carry out the dissertation work. His constant support and interest in the subject equipped me with a great understanding of different aspects of the project work.

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I am grateful to my **Parents** for their love & support throughout my life. Thank you both for giving me strength to reach for the stars & chase my dreams. My Big brother **Tarak**, **Bhabhi & Sisters** deserve my wholehearted thanks as well.

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Abbreviation Notation and Nomenclature

PCC	Plain Cement Concrete
FRC	Fibre Reinforced Concrete
RCC	Reinforced Cement Concrete
NSC	Normal Strength Concrete
HSC	High Strength Concrete
RC	Reinforced Concrete
M25(PCC)	Plain Cement Concrete Mix of M25 Grade
M60(PCC)	Plain Cement Concrete Mix of M60 Grade
M25(FRC)	Fibre Reinforced Concrete Mix of M25 Grade
M60(FRC)	Fibre Reinforced Concrete Mix of M60 Grade
°C	Temperature in Degree Celsius
LVDT	Linear Variable Differential Transducer
MoE	Modulus of Elasticity
A_{st}	Area of steel reinforcement
f_{ck}	Characteristic compressive strength of concrete
f_y	Characteristic yield strength of main steel
P_u	Design load of column
d'	Effective cover
l_x	Effective length along X-direction
l_y	Effective length along Y-direction
D_x	Depth along X-direction
D_y	Depth along Y-direction
$e_{x\ min}$	Minimum eccentricity in X-direction
$e_{y\ min}$	Minimum eccentricity in Y-direction
M_{ux}	Moment due to minimum eccentricity in X-direction
M_{uy}	Moment due to minimum eccentricity in Y-direction
p	Percentage of steel required
M_{ux1}	Maximum uniaxial moment capacity for an axial load of P_u bending @ X-axis
M_{uy1}	Maximum uniaxial moment capacity for an axial load of P_u bending @ Y-axis

Chapter 1

Introduction

1.1 General

Concrete is possibly exposed to elevated temperatures during exposed to fire or when it is near to furnaces and reactors. Physical properties such as Change in colour, Change in volume, spalling & weight loss are significantly visible at elevated temperature. Mechanical properties such as Compressive strength, Flexural strength, split tensile strength, modulus of elasticity & bond strength significantly reduces at elevated temperature. Change in Physical & Mechanical properties may result in undesirable structural failures. Therefore, the properties of concrete retained after a fire are of still importance for determining the load carrying capacity and for reinstating fire-damaged constructions procedure. The chemical composition and physical structure of the concrete change considerably when exposed to elevated temperature. Dehydration such as the release of chemically bound water from the calcium silicate hydrate (CSH) becomes significant above about 110 °C. The dehydration of the hydrated calcium silicate and the thermal expansion of the aggregate increase internal stresses and from 300 °C micro-cracks are induced through the material. Calcium hydroxide [$Ca(OH)_2$], which is the most important compounds in cement paste, dissociates at around 530 °C results in the shrinkage of concrete. The fire is generally extinguished by water and CaO turns into [$Ca(OH)_2$] causing cracking of concrete. Thus, the effects of high temperatures are generally visible in the form of surface cracking and spalling. The alterations produced by high temperatures are more evident when the temperature rises beyond 500 °C. Most changes experienced by concrete material at this temperature level are considered irreversible in nature. CSH gel, which is the strength providing compound of cement paste, decomposes above 600 °C. At 800 °C, concrete is usually crumbled and above 1150 °C feldspar constituent melts and the other minerals of the cement paste turn into a glass phase. As a result, severe micro-structural changes are induced and concrete loses its strength and durability aspects.

Figure 1.1 & 1.2 represent fire damaged residential structure during February 2016, located in (Ranip)Ahmedabad.

Figure 1.1: Fire Exposed Building(Saket Apartment,Ahmedabad)

Figure 1.2: Damage Due to Fire (Saket Apartment,Ahmedabad)

1.2 Need of Study

Concrete is well known for its inherent fire resistive property but at elevated temperatures it undergoes chemical and physical modifications. Chemical modification includes dissociation of CSH gel & $Ca(OH)_2$ which are most important binding compounds of concrete. Physical modifications include thermal expansion of aggregates, crack initiation, spalling effects, weight loss, etc. Such modifications are responsible for degradation in mechanical properties of concrete.

The proposed work is to study the behavior of Normal Strength Concrete(NSC) & High Strength Concrete(HSC) exposed to different elevated temperatures such as i.e. 300°C, 500°C, 700°C & 900°C, respectively. Comparison of mechanical properties & physical properties of Normal strength concrete & High strength concrete with 4 different kinds of concrete mixes namely M25(PCC), M25(FRC), M60(PCC) & M60(FRC) are required to be made at different elevated temperatures, respectively.

An Attempt is made to acquire the behaviour of all 4 kind of concrete mixes under the axial compressive load after exposed to extreme elevated temperature.

1.3 Objectives of Study

To study various parameters, following objectives are decided for the major project.

- To study the physical behavior of Plain Normal strength concrete & high strength concrete exposed to different elevated temperatures i.e.300°C, 500°C, 700°C & 900°C, respectively.
- To evaluate the change in performance of normal strength concrete & high strength with incorporation of steel fibre exposed to high temperature.
- To Compare mechanical properties such as compressive strength, split tensile strength, flexural strength, modulus of elasticity & bond strength etc for different concrete mixes exposed to elevated temperatures.
- To study the behavior of RC columns of different concrete mixes exposed to extreme elevated temperatures. The study includes parameters like ultimate failure load, load v/s displacement relationship, axial stress v/s strain relationship, failure modes, crack patterns, etc for all columns.

1.4 Scope of Work

Scope of work of major project includes theoretical work & laboratory work related to mechanical & physical properties of PCC, FRC & RC elements of various types of concrete.

Plain Cement Concrete & Fiber Reinforced Concrete Mixes:

Mechanical properties such as compressive strength, flexural strength, split tensile strength, modulus of elasticity & bond strength of 4 different concrete mixes are to be measured after exposed to different elevated temperature i.e. 300°C, 500°C, 700°C, 900°C respectively. Four different mixes namely Normal Strength concrete [M25(PCC) & M25(FRC)] & High strength concrete [M60(PCC) & M60(FRC)] are incorporated in this study. Steel fibre dosage adopted as 1% by volume of concrete for fibre reinforced concrete mixes.

Concrete specimens of all concrete mixes are to be cast & water cured for duration of 28 days. After curing, the specimens are to be allowed to air-dry for 1 day. After that the specimens are to be exposed to different elevated temperatures in gas fired furnace. After heating specimens for duration of 1 hour at target temperature, the specimens are allowed to air-cool at room temperature till steady state condition is achieved. This procedure is followed by destructive testing to evaluate residual mechanical properties for all concrete mixes. Residual mechanical properties of heated specimens are to be compared with that of unheated specimens to calculate the relative strength. Mechanical properties includes Compressive strength, Flexural strength, Split tensile strength, Modulus of Elasticity & Bond strength. Average results of three specimens are to be taken as a final result.

The detail of tests to be conducted on specimens, dimension of specimens and various temperatures at which the specimens are to be heated have been presented in Table 1.1. Entire scope of work has been presented in form of a flowchart in Figure 1.3.

Table 1.1: Tests Performed on PCC & FRC Elements

Test Name	Unheated	300°C	500°C	700°C	900°C
Compression Test (IS:516)[19] 150 mm × 150 mm × 150 mm	3	3	3	3	3
Flexure Test (IS:516)[19] 100 mm × 100 mm × 500 mm	3	3	3	3	3
Split Tensile Test (IS:5816)[20] 150 mm × 300 mm	3	3	3	3	3
Modulus of Elasticity (IS:516)[19] 150 mm × 300 mm	3	3	3	3	3
Bond Strength (IS:516)[24] 150 mm × 150 mm × 150 mm	3	3	3	3	3

RCC Columns:

RC columns of different concrete mixes such as M25(PCC), M25(FRC), M60(PCC) & M60(FRC) are to be cast and tested as mentioned in Table 1.2. Two columns of each category of concrete mixes are to be cast and to be tested.

Ultimate failure load of RC column is to be designed as per IS:456(2000)[30]. Test is to be conducted under axial compressive force in loading frame for all the columns. Cross-section dimension, length, diameter of reinforcement, numbers of bars etc for RC column are to be calculated. The columns of size 150 × 150 × 1000 mm are to be cast & water cured for 28 days. After curing period, columns of all mixes are exposed to 900°C temperature for duration of 1 hour at target temperature. Mechanical properties such as Residual axial load carrying capacity, displacement, stress, strain for all categories of columns are to be evaluated and to be compared with that of unheated column specimens. Average results of two RC columns are to be taken as a final result.

Table 1.2: List of RC Columns for Different Mixes

Concrete Type	Element	Dimensions(mm)	Unheated (Nos.)	At 900°C	Total
M25 (PCC)	RC Column	150 × 150 × 1000	2	2	4
M25 (FRC)	RC Column	150 × 150 × 1000	2	2	4
M60 (PCC)	RC Column	150 × 150 × 1000	2	2	4
M60 (FRC)	RC Column	150 × 150 × 1000	2	2	4

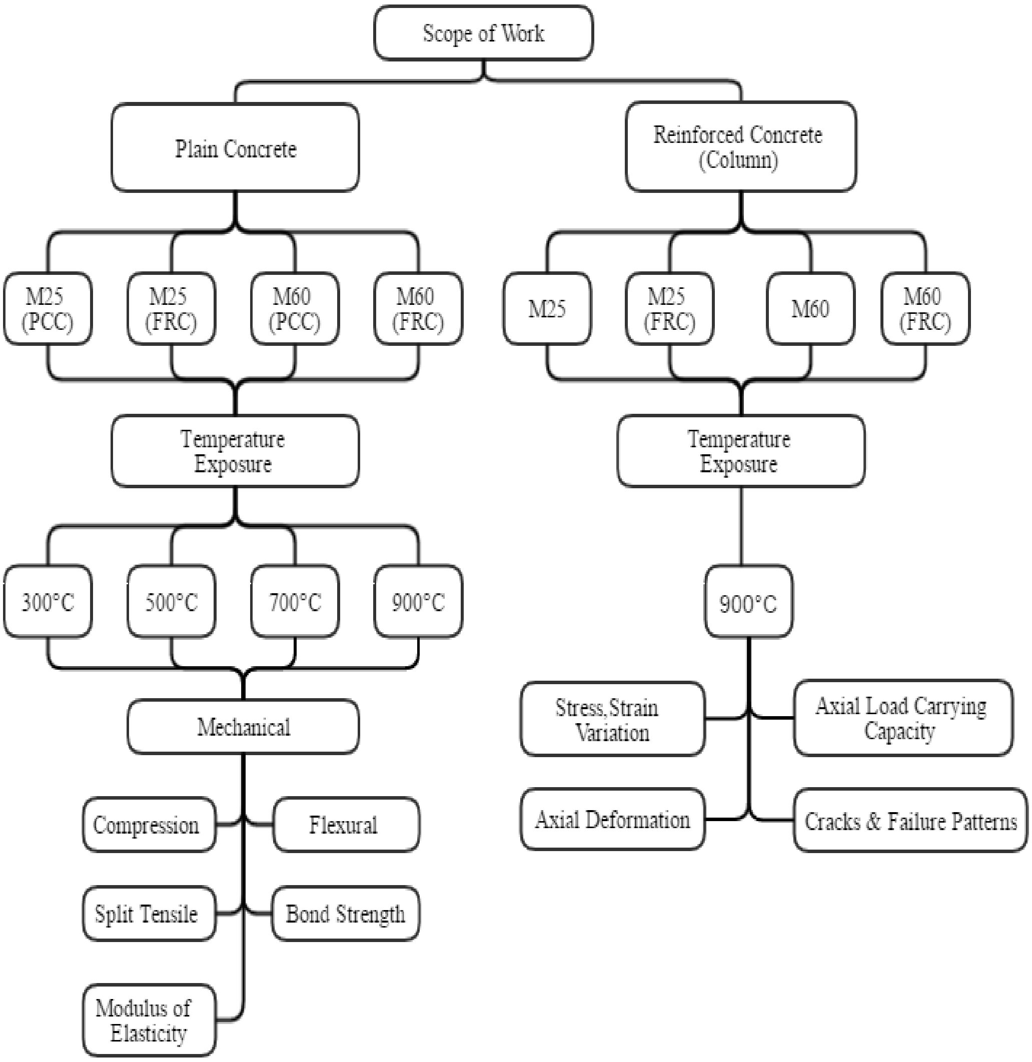


Figure 1.3: Flow Chart of Scope of Work of Major Project

1.5 Organization of Major Project

This study is related to Residual mechanical and physical properties of Plain concrete elements and Reinforced concrete elements (RC columns with same configuration of all category of concrete mixes) subjected to different elevated temperatures. Brief overview of each chapter and relevant contents has been explained briefly as mentioned below.

Chapter 1 deals with introduction, Need of study & objectives of study. The Scope of work has also been explained in this chapter.

Chapter 2 discusses literature review. Many researchers have worked upon different types of concrete subjected to elevated temperatures have been included in this chapter.

Chapter 3 deals with details of experimental programme which includes material properties, Mix design, testing procedures of specimens subjected to elevated temperature & column design with testing procedure employed during the investigation.

Chapter 4 explains behavior of plain concrete elements (PCC & FRC) subjected to different elevated temperatures i.e. 300°C, 500°C, 700°C & 900°C. It deals with relative mechanical properties and physical changes of plain concrete elements at different elevated temperatures. Residual mechanical strengths have been compared with that of unheated specimens in this chapter.

Chapter 5 deals with behavior of RC column members subjected to extreme temperature of 900°C. The results such as Ultimate failure load, deflection,stress, strain, failure patterns and cracks for all columns have been represented in this chapter. Axial load carrying capacity of heated specimens have been compared with unheated specimens over here.

Chapter 6 consists of summary, conducting remarks and recommendation for future scope of work on basis of work in the major project.

Chapter 2

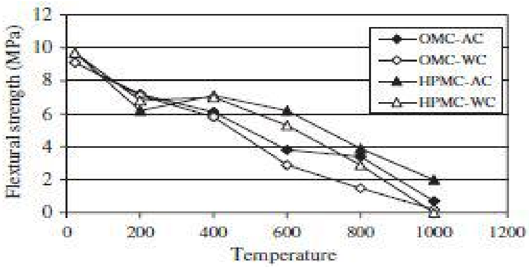
Review of Literature

2.1 General

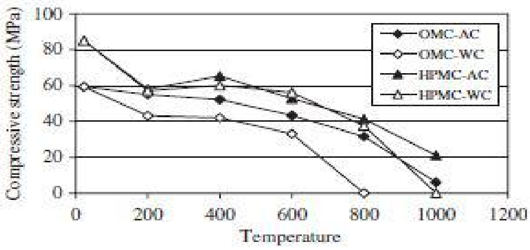
Concrete structures are sometimes exposed to the effects of fire. Although concrete is capable of resisting high temperature,at extreme high temperatures it starts loosing it's strength in terms of mechanical properties. Many researchers have carried out an investigation on behavior of different kind of concrete mixes exposed to different elevated temperature with different kind time-temperature relationships. Following literatures represents behavior of plain concrete mixes & reinforced concrete elements during elevated temperature exposure. Residual mechanical properties i.e. compressive strength, Split tensile strength, Flexure test & bond strength have been incorporated in section of Plain concrete elements with variables as concrete mix type, Temperature range, Exposure duration & time-temperature relationships.

2.2 Investigation on Plain Concrete Elements

Husem[1] carried out an experimental work on ordinary micro-concrete & high performance micro concrete, subjected to different elevated temperature in order 200°C, 400°C, 600°C, 800°C, 1000°C for a specific time period to measure residual Compressive strength & Flexural strength under 2 different cooling regime. OMC & HPMC both looses strength (in flexure and compression) with increase in temperature but specimens cooled with water looses strength rapidly than cooled in air as presented in Figure 2.1. Strength gain is achieved during range of 200-400°C due to evaporation of free water by means of hydration process during that temperature range.



(a) Loss in Flexural Strength with Increase in Temperature



(b) Loss in Compressive Strength with Increase in Temperature

Figure 2.1: Degradation in Mechanical Properties along with Rise in Temperature

Rao & Kumar[2] carried out an experimental work to find out mechanical properties of normal strength concrete & high strength concrete exposed to elevated temperature. They carried out testing for degradation of compressive strength, loss in weight, change in colour, spall of concrete for various temperatures 200° C, 400°C, 600°C, 800°C and for different cooling regimes i.e. Air cooling & water cooling. Total Specimens were subjected to different elevated temperature for 60 minutes. During exposed to elevated temperature, HSC is more vulnerable compare to NSC.Air cooling results shows highest strength degradation loss as presented in Table 2.1.

Table 2.1: Residual Compressive Strength

S No	Temp. °C.	Air cooling			Water quenching			Hot condition		
		M20	M40	M60	M20	M40	M60	M20	M40	M60
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
	At room temp	25	48	65	25	48	65	25	48	65
1	200	26 (0)	43 (10.4)	53.45 (17.8)	23.16 (7.4)	40.25 (16.2)	50.6 (22.2)	26.12 (0)	41.6 (13.4)	54.81 (15.7)
2	400	25.33 (0)	29.16 (39.3)	42.12 (35.2)	22 (12)	34.16 (28.9)	45.8 (29.6)	25 (0)	42 (12.5)	47.45 (27)
3	600	20.62 (17.6)	18.15 (62.2)	28.5 (56.2)	21.41 (14.4)	26.12 (45.6)	33.2 (49)	18.21 (27.2)	33.81 (29.6)	34.6 (46.8)
4	800	10.15 (59.4)	13.8 (71.2)	16 (75.4	18.12 (27.6)	15.12 (68.5)	20.4 (68.7)	14.16 (43.4)	18.18 (62.2)	23.61 (63.8)

Poon et al.[3] carried out an experimental study on compressive behaviour of high performance concrete produced with incorporation of different mineral admixture (metakaoline & silica fume) & different fibers with variable dosage, subjected to elevated temperatures of 600°C & 800°C. Steel fibers of 25 mm length and 60 aspect ratio. Polypropylene fibers of 19 mm length and 360 aspect ratio. 1% steel fiber incorporation showed higher residual strength as presented in Table 2.2.

Table 2.2: Residual Compressive Strength of Different Mixes for Different Temperatures

Mix type	Fiber type & Dosage	Comp. strength (Unheated) (MPa)	Comp. Strength heated at 600 °C (MPa)	Comp. Strength heated at 800 °C (MPa)
PC-0	No fiber	69.1	32.79	17.64
PC-1	1% steel	71.4	38.95	23.8
PC-2	0.22% PP	68.5	34.31	17.07
PC-3	1% steel + 0.22% PP	69.6	36.19	22.53
MK-0	No fiber	86.1	33.28	15.76
MK-1	1% steel	87.5	38.72	21.41
MK-2	0.11% PP	86.1	35.1	14.98
MK-3	0.22% PP	84.6	31.69	14.49
MK-4	1% steel + 0.22% PP	86	38.65	17.81
SF-0	No fiber	82.8	37.84	20.55
SF-1	1% steel	83.7	39.19	23.46
SF-2	0.11% PP	81.8	36.85	15.55
SF-3	0.22% PP	81.2	33.76	14.09
SF-4	1% steel + 0.22% PP	82.9	37.69	19.25

Lau & Anson[4] investigated a mechanical properties of (NSC)Normal strength concrete & High strength concrete (HSC),With & Without incorporation of Steel Fibers, exposed to different elevated heating temperature ranging between 105°C & 1200°C. Experimental program consisted of casting & testing of Six different mixes, In which M-1,M-2,M-3 are Plain cement concrete and M-1F,M-2F,M-3F are fiber reinforced cement concrete having 28 days compressive strength as 39 MPa,53 MPa,99 MPa,45 MPa,60 MPa,110 MPa, respectively. Proportion steel fiber added was 1% of concrete volume,having 25 mm length,0.4 mm dia(aspect ratio=62.5).Test results shows that High strength concrete is more vulnerable compare to the normal strength concrete beyond 600°C and losses significant amount of compressive strength. SFRC specimens after being subjected to extreme elevated temperature suffers severe shape deformation compare to non-fibre concrete. However Non-fibre concrete shows severe cracking at extreme temperatures.Result shows that concrete mixes with incorporation of steel fibres shows lesser loss in Modulus of Elasticity compare to mixes without steel fibre as presented in Figure 2.3. Figure 2.2 shows Time-temperature Curves.

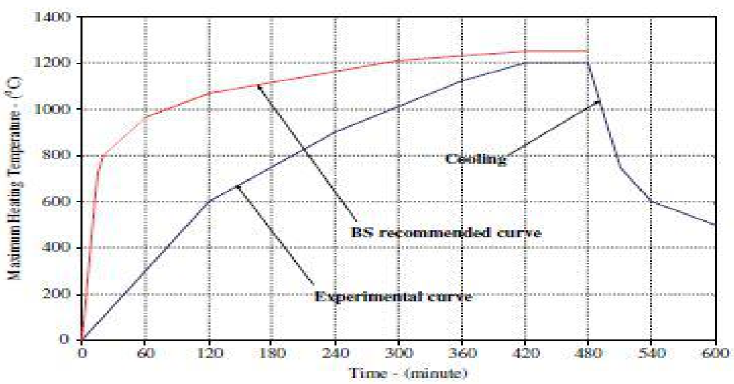


Figure 2.2: Graphical Representation of Time-Temperature Curve

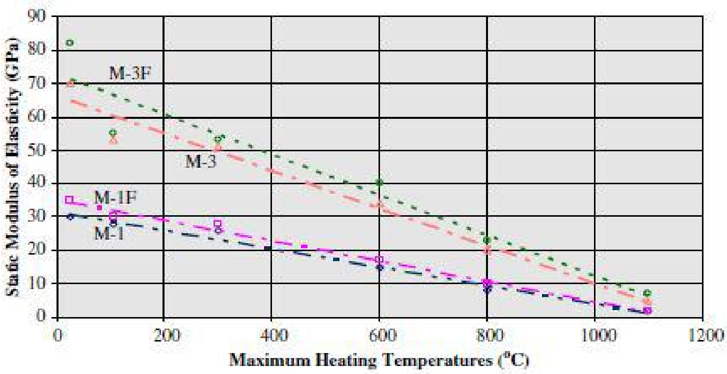
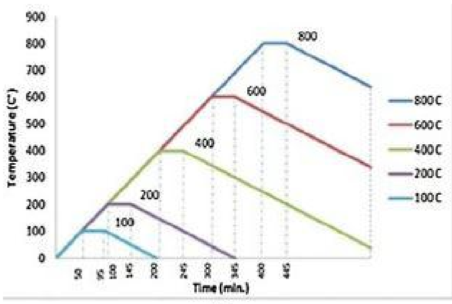
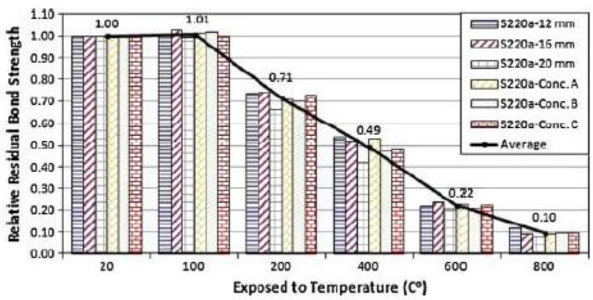


Figure 2.3: Loss in Modulus of Elasticity for Different Mixes at Elevated Temperatures

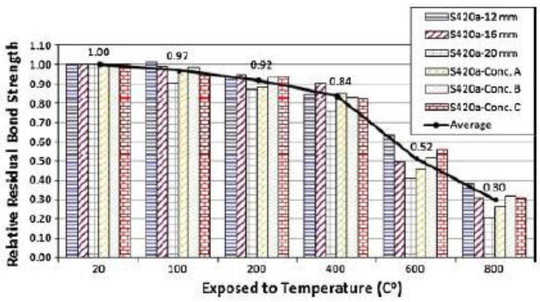
Ergun et al.[5]carried out an experimental research on residual bond strength comprises of 3 different concrete mixes (concreteA-20 MPa, ConcreteB-34 MPa, ConcreteC-44 MPa) with 3 different steel dia. bars (12mm,16mm,,20mm) with 3 different yield strength (S220a,S420a,S500a) exposed to different elevated temperatures (200°C, 400°C, 600°C, 800°C). Specimen for bond strength were having 150 mm dia.,300 mm height with 250 mm of embedded bar length. Samples were heated in furnace at a rate of 2°C/min and were kept exposed for 45 minutes at elevated temperature. Residual bond strengths for different configurations have been represented in Figure 2.4.



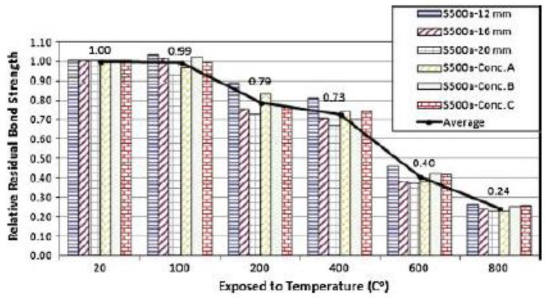
(a) Heating Regimes



(b) Bond Strength Degradation for S220a Bars



(c) Bond strength degradation for S420a Bars



(d) Bond strength degradation for S500a Bars

Figure 2.4: Bond Strength Degradation for Different Steel Bars

Bastami et al.[6]investigated a effect of elevated temperature on High strength concrete. Research programme consisted of 16 different high strength concrete mixes made with 4 variables namely w/c ratio, sand ratio, silica fume ration and amount of silica fume added. Specimen dimensions were of 150mm×300mm cylinder, which were water cured for 28 days and exposed to elevated temperature at a increment rate of 20°C/min as presented in Figure 2.5. Different mix proportions & residual mechanical properties have been presented in Table 2.3 & 2.4 respectively.

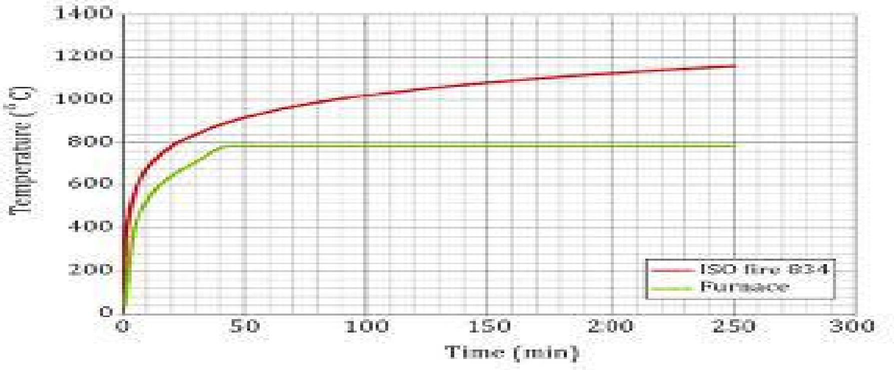


Figure 2.5: Graphical Representation of Experimental Curve & ISO-834 Curve

Table 2.3: Description of Concrete Mixtures

Mixture	Water (kg)	Cement (kg)	Silica fume		Water/ binder	Coarse aggregate		Fine aggregate		Superplasticizer	
			Mass (kg)	Percent of binder mass (%)		Mass (kg)	Percent of total aggregate amount (%)	Mass (kg)	Percent of total aggregate amount (%)	Mass (kg)	Percent of binder mass (%)
M1	151	520	64	11.0	0.26	1153	66	600	34	11	1.9
M2	170	567	–	–	0.30	955	59	653	41	7	1.2
M3	130	513	43	7.7	0.23	1080	61	685	39	16	2.9
M4	144	564	89	13.6	0.22	1068	64	593	36	20	3.1
M5	125	545	52	8.7	0.21	1103	65	589	35	18	3.0
M6	135	500	30	5.7	0.25	1110	61	700	39	14	2.6
M7	158	568	–	–	0.28	1068	63	617	37	12	2.1
M8	151	519	62	10.7	0.26	1122	66	579	34	19	3.3
M9	165	535	80	13.0	0.27	1153	66	597	34	25	3.9
M10	150	500	–	–	0.30	927	55	758	45	3	0.6
M11	150	500	25	4.8	0.29	1004	55	822	45	9	1.7
M12	150	500	75	13.0	0.26	1016	57	760	43	23	4.0
M13	150	500	25	4.8	0.29	1233	67	617	33	8	1.5
M14	150	475	24	4.8	0.30	1066	60	710	40	13	2.6
M15	156	486	47	8.8	0.29	1068	61	676	39	11	2.1
M16	150	500	–	–	0.30	1234	67	617	33	8	1.6

Table 2.4: Relative Compressive Strength after Exposed to Fire

Mixture	Strength at room temperature (MPa)	Residual strength after fire exposure (MPa)	Relative strength (%)
M1	76.7	20.6	26.8
M2	66.0	23.9	36.2
M3	85.7	15.9	18.5
M4	82.3	14.9	18.0
M5	77.5	20.6	26.6
M6	90.1	18.2	20.2
M7	65.8	9.5	14.5
M8	93.6	16.1	17.1
M9	80.3	13.9	17.3
M10	65.2	–[a]	–
M11	78.5	19.5	24.9
M12	82.3	27.3	33.2
M13	81.4	21.0	25.8
M14	76.2	16.2	21.3
M15	64.5	13.9	21.6
M16	71.8	–[a]	–

Ergun et al.[7]investigated a mechanical properties (compressive strength & Flexural strength) of specimens produced of 2 different mixes subjected to different elevated temperatures of 100°C, 200°C, 400°C, 600°C & 800°C. Major objective of the study was to check, whether at elevated temperature different dosage of cement content affects the residual mechanical properties as presented in Table 2.5. Specimens were heated as per given beloved heating regime as presented in Figure 2.6. Experimental result shows that, there is no any significant effect of cement dosage on residual mechanical properties of concrete mixes at elevated temperature as presented in Figure 2.7.

Table 2.5: Concrete Mix Proportions(kg/m^3)

Series	w/c	Cement	Water	River sand 0–4 mm	Crushed sand stone 0–6 mm	Crushed stone II 6–12 mm	Crushed stone III 12–22 mm
I	0.5	250	123	417	407	425	849
II	0.5	350	172	373	365	381	761

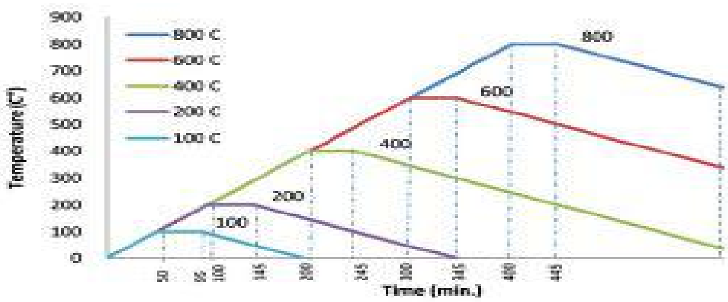
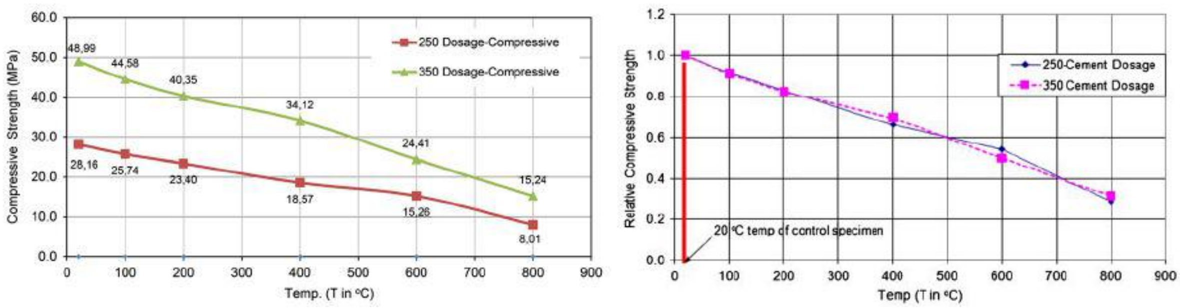


Figure 2.6: Heating Regimes



(a) Degradation in Compressive Strength (b) Degradation in Flexure Strength

Figure 2.7: Loss in Mechanical Properties

Arioz[8]carried out an experimental investigation on 4 different concrete mixes with a variables of coarse aggregate type (crushed stone aggregate & river gravel aggregate) and w/c ratio subjected to elevated temperatures ranging between 200°C-1200°C as presented in Table 2.6. Cubes of sizes 70mm×70mm×70mm were cast and water cured for 28 days duration. Temperature increment rate was 20°C/min & exposure duration was 2 hours. Test result shows that there is no significant effect of w/c ratio on mechanical properties of concrete at elevated temperatures. At elevated temperature Crushed lime stones retains higher strength compare to river gravel aggregates as presented in Figure 2.8.

Table 2.6: Mix Design Proportion & 28 Days Compressive Strength Results

Mixture	Mix proportions (kg/m^3)				Some properties		
	Coarse aggregate	Fine aggregate	Cement	Water	w/c ratio	Type of aggregate	Compressive strength[a] (MPa)
CL05	716	716	500	250	0.5	Crushed limestone	52
CL06	653	653	500	300	0.6		44
RG04	819	819	500	200	0.4	River gravel	52
RG05	753	753	500	250	0.5		39

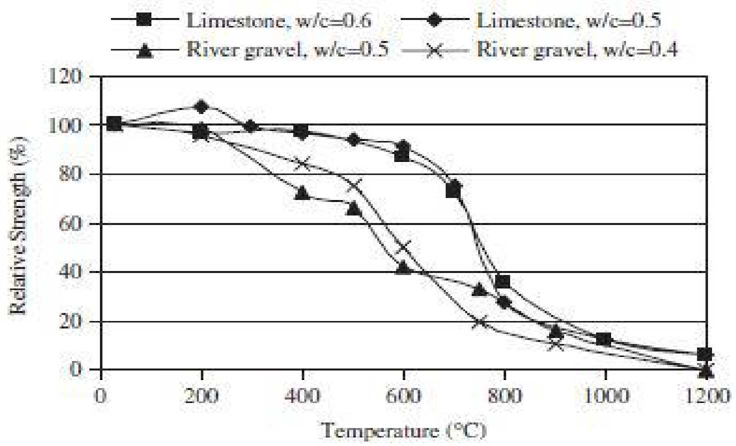


Figure 2.8: Relative Compressive Strength of Concrete Mixes after Exposed to Elevated Temperatures

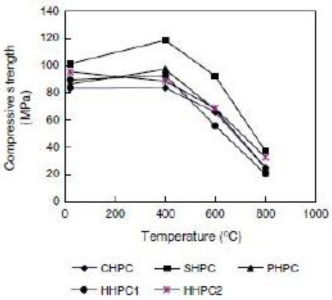
Peng et al.[9]investigated 5 different kinds of high performance concrete mixes incorporating different fiber dosages exposed to different elevated temperatures ranging between 200° to 800°C as presented in 2.7. Exposure duration was 60 minute at target temperature where heating rate was 10°C/min. Specimen sizes were 100mm×100mm×100mm for compression test and 100mm×100mm×300mm for splitting tensile test. Residual mechanical properties were found out namely compressive strength, tensile splitting strength and fracture energy as presented in Figure 2.9. Results shows that fiber incorporation enhanced the resistance against spalling and fracture energy.

Table 2.7: Mix Proportions & 28 Days Compressive Strength Results

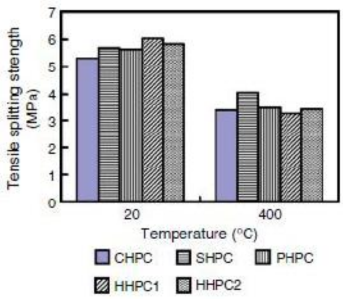
Mix proportion and 28-day compressive strength*

Type	*W/B*	Quantity (kg/m^3)							Strength (MPa)/ density (kg/m^3)
		C	SF	S	G	W	PP-F	S-F	
CHPC	0.26	535	64	597	1153	156	0	0	93.9/2506
PHPC	0.26	519	62	579	1122	151	1	0	80.6/2434
HHPC1	0.26	527	63	588	1138	154	0.6	40	109.6/2511
HHPC2	0.26	528	63	590	1140	154	0.3	70	109.2/2545
SHPC	0.26	518	62	578	1119	151	0	100	109.3/2528

* *W/B* for water/binder ratio (mass), C for ordinary Portland cement (OPC), SF for silica fume, S for sand, G for coarse aggregate, W for water, PP-F for 20 μm polypropylene fiber (PP fibre), and S-F for steel fiber, respectively.



(a) Loss in Compressive Strength



(b) Loss in Split Tensile Strength

Figure 2.9: Degradation in Mechanical Properties

2.3 Investigation on Reinforced Concrete Elements

Kodur et al.[10]investigated a behavior of NSC & HSC columns exposed to elevated temperature. Experiment consist of 5 different concrete mixes consisted of NSC , HSC and HSC with fiber incorporation as presented in Table 2.8. Test specimens were having dimensions of 3810 mm length and square cross-section of 305 mm length. 4 Nos. of 25 mm dia. bar,tied with 10 mm dia. bar at a spacing of 75 mm c/c at ends and 145 mm c/c in the middle. Loading was given during the fire exposure. Result shows that HSC column have lower fire resistance compared to NSC as presented in Table 2.9. Polypropylene fiber improves resistance against spalling.

Table 2.8: Mix Proportion & Properties of Concrete Mixes

	Batch (Specimen Type)				
Property	Mix 1	Mix 2	Mix 3	Mix 4	Mix 5
Column	TNC1	THC4	THC8	THS10	THP13
Cement content (kg/m^3)	355	483	483	483	483
Fine aggregate (kg/m^3)	949	669	667	721	721
Coarse aggregate (10 mm) (kg/m^3)	732	1,040	1,040	900	900
Aggregate type	Siliceous	Siliceous	Carbonate	Siliceous	Siliceous
Water (kg/m^3)	258	177	175	184	182
Water-binder ratio	0.73	0.38	0.37	0.39	0.39
Fiber (kg/m^3)	0	0	0	42	0.9
Fiber type	—	—	—	Steel	Polypropylene
Silica fume (kg/m^3)	0	42	42	42	42
Superplasticizer (kg/m^3)	0	20	19	18	20
28 day compressive strength (MPa)	28	61	60	63	52
90 day compressive strength (MPa)	40	100	73	89	87

Table 2.9: Summary of Test Parameters & Results

		Concrete Strength (fc)					
Column	Column dimensions (mm)	28 day (MPa)	Test day (MPa)	Factored resistance (Cr) (kN)	Test load (C) (kN)	Load intensity (C/Cr)	Fire resistance (h:min)
TNC1	305×305	27.8	40.2	1,728	930	0.54	4:38
THC4	305×305	60.6	99.6	3,697	2,000	0.54	3:24
THC8	305×305	60.4	72.7	2,805	2,000	0.71	5:05
THS10	305×305	63.2	89.1	3,349	1,800	0.54	3:59
THP13	305×305	51.9	86.6	3,266	1,800	0.54	4:31

Jau & Huang[11]investigated behavior of corner column under axial loading for 2 hours & 4 hours fire exposure scenario. Specimen dimensions of 300×450×2700 mm were cast, with 27.6 MPa as concrete strength & 413.8 MPa as rebar strength. Results shows that the core dimensions of columns affected residual strength. Residual strength ratios of the columns after fire loading for 2h & 4h were 67% & 57% accordingly.

Lin & Tsay[12]investigated experimentally & analytically the deterioration in strength & stiffness of RC column after exposed to fire. Total 25 Nos. of columns were considered for fire exposure at different durations of fire. Columns were tested under concentric axial loading & eccentric axial loading. Columns having three different cross sections were considered namely 40cm×40cm, 30cm×30cm & 20cm×20cm. Each columns were having different tie spacing configuration. Experimental results shows that duration of fire have a significant effect on residual stiffness & strength of column. Size of the column have significant effect as size of column increases, the deterioration in strength & stiffness decreases.

Raut & Kodur[13]investigated a response of High strength concrete columns under design fire exposure scenario. Columns of 6 different mixes cast having dimensions 203×203×3350 mm. Columns were having main reinforcement bars of 20mm & stirrups of 10mm at 200 mm c/c. 2 columns were exposed as per ASTM time-temperature relationships while remaining columns were tested under design fire relationships as represented in Figure 2.10. Experimental results shows that resistance of fire for HSC columns decreases as much as 65% to that of NSC columns. Incorporation of polypropylene fibres reduces spalling effect. Type of fire exposure has a significant effect on residual properties of HSC concrete columns.

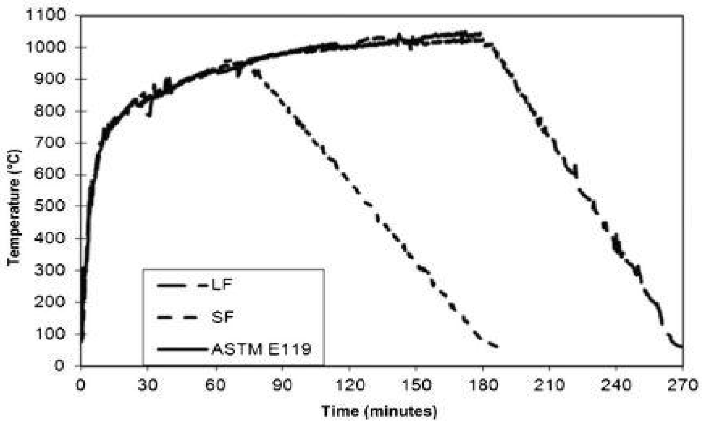


Figure 2.10: Different Time-Temperature Curves Adopted for Experimental Study

Tan & Nguyen[14]investigated a Structural responses of reinforced concrete columns subjected to uni-axial bending and restraint at elevated temperatures. Total 6 Nos. of specimens were tested to failure level to investigate the effects of uni-axial bending, axial restraint and initial load level on the responses of reinforced concrete columns exposed elevated temperatures. Dimensions of columns were 300×300×3300 mm. Temperature dependent axial deformations, lateral deflections failure modes, and failure times of test specimens were compared with that of analytically obtained results. Experimental results shows that lateral deflections at elevated temperature is adversely affected by eccentricity & initial load level.

2.4 Findings from Review of Literature

Review of Literature have been carried out consisting of different variable such as Normal strength concrete & High strength concrete, plain cement concrete & fibre reinforced concrete, different temperatures, different heating regimes, different heating rates, different mechanical properties(compressive strength, flexure strength, split tensile strength, bond strength & MoE). Experimental results shows that as temperature increases loss in mechanical strength higher. High strength concrete suffers more as compare to Normal strength concrete. Fibre reinforced concrete have higher residual strength as compare to plain cement concrete. Out of different fibres, steel fibres are more effective to resist elevated temperatures. Heating regime have a significant effects on residual mechanical strength of concrete. RC column produced from HSC presents high amount of strength & stiffness degradation as compare to NSC. Fibre incorporation in RC column are beneficial to resist adverse effect due to elevated temperature.

Chapter 3

Experimental Programme

3.1 General

In this chapter, material properties, mix design procedures adopted, method of casting employed have been discussed. Methods of evaluation of mechanical properties of PCC elements have been explained. Design of RC column as per IS codes, test setup of the specimens, test procedures and test parameters are covered.

3.2 Material Properties

Materials used for the experimental investigation are processed fly ash as source material, Ordinary Portland Cement, aggregates, gypsum, water and admixture.

3.2.1 Cement

53 grade ordinary Portland cement is used for the experimental work. The physical and chemical properties are given by cement manufacturing company. The chemical and physical properties of cement are presented in Table 3.1 and Table 3.2 respectively.

Table 3.1: Physical Properties of Cement

Sr. No.	Properties	Results obtained	Specifications (IS: 12269-1987)[25]
1	Compressive Strength (MPa)		
	3 days	29.17	27 (min)
	7 days	40.02	37 (min)
	28 days	55.19	53 (min)
2	Fineness (m^2/kg)	309	225 (min)
3	Setting Time (minute)		
	Initial Setting time	125	30 (min)
	Final setting time	218	600 (max)
4	Soundness		
	Le-chatelier (mm)	1	10 (max)
	Autoclave (%)	0.13	0.8 (max)

Table 3.2: Chemical Properties of Cement

Sr. No.	Properties	Results Obtained	Specifications (IS: 12269-1987)[25]
1	Loss on ignition (%)	1.81	4 (max)
2	Sulphuric Anhydride (%)	2.77	3.5 (max)
3	Magnesia (%)	3.6	6 (max)
4	Insoluble Residue (%)	0.95	2 (max)
5	Chloride Content (%)	0.045	0.10 (max)
6	Lime Saturation Factor (%)	0.92	0.80 to 1.02
7	Alumna Iron Ratio (%)	1.25	0.66 (min)

3.2.2 Fly Ash

Pulverised Fuel fly ash is used in present experimental investigation. The source of fly ash is the thermal power plant at Ankleshwar, Gujarat. The properties of fly ash as given by the manufacturer is presented in Table 3.3. The fly ash is classified siliceous pulverized fuel fly ash as per IS:3812 (Part-1)[23].

Wet Sieve Analysis

Wet sieving test is conducted for evaluating the percentage of material passing 45-micron sieve. In this test, 100 gm of fly ash is taken in 45-micron sieve. The material is washed with a jet of water and keep it well agitated. The washing is continued till it appears no more turbid. After washing of sieve, the sieve is allowed to dry in an oven with residue. The residue from the sieve after drying, is weighed on a balance sensitive to 0.1 percent of the weight of the test sample. The percentage of material passing sieve on wet sieving is reported to the nearest 0.1 percent by weight of the test sample. Allowable percentage retained on 45-micron sieve is 34 percent as IS:3812 (Part-1)[24].
Results of wet sieve analysis is mentioned below:

- Size below 45 micron sieve : 82.37%
- Size above 45 micron sieve : 17.63%

Pozzolanic Activity Index

The main purpose of this test is to check the effect of fly ash as pozzolona when 20% of cement is replaced with fly ash. When cement mortar with fly ash is compared with control mortar, minimum achievable strength of cement mortar with fly ash on 28-day is required to be 80%. It means addition of fly ash is not interfering with the hydration chemistry of OPC. Pozzolanic activity index is a ratio of strength of fly ash blended mortar and strength of control mortar. In control mortar, materials i.e. cement and ennore sand are taken as 225 gms and 675 gms, respectively and are mixed thoroughly well. Water is added to this mix till thixotropic point. 50 mm cubes are cast with this mix in two layers thoroughly pressing with the thumb. Mould is tamped for minimum 5 times for better consolidation of the matrix. In this fly ash blended mortar, 20% OPC is replaced with fly ash. Three cubes are made of both type of mortar. Cubes are tested at the age of 28 days to evaluate compressive strength of both types of mortar. Pozzolonic activity index is determined by following equation.

$$\text{Pozzolanic Activity Index (PAI)} : \frac{\text{Strength of Fa blended mortar}}{\text{Strength of control mortar}} \times 100 \tag{3.1}$$

Results of PAI is derived below:
Pozzolanic Activity Index (PAI): $\frac{49.09}{55.64} \times 100$
Pozzolanic Activity Index (PAI): 88.23%

Properties of fly ash have been presented in Table 3.3.

Table 3.3: Properties of Fly Ash

Sr.No.	**Test Details**	**Unit**	**Test Results**	**Requirements of Siliceous Pulverised Fuel Fly Ash**
1	Colour	-	Light Grey	Grey
2	Specific surface Area	m^2/kg	332.94	Min 320
3	Loss on Ignition	N/mm^2	1.05	Max 5%
4	$SIO_2 + Al_2O_3 + Fe_2O_3$	%	93.02	Min 70% by mass
5	SIO_2	%	61.40	Min 35% by mass
6	MgO	%	1.42	Max 5% by mass
7	SO_3	%	0.56	Max 3% by mass
8	Na_2O	%	0.62	Max 1.5% by mass
9	Total chlorides	%	0.03	Max 0.05% by mass

3.2.3 Silica Fume

Silica fume is a highly reactive pozzolanic mineral additive in powder form to make durable concrete with higher compressive strength and low permeability having better finish. Its particle size is smaller than cement and fly ash particles. IS-456:2000 recommends use of silica fume is mineral admixture & the same is governed under ASTM-618 C-class N-Pozzolana. The particle size of silica fume is significantly smaller than cement & blending leads to enhance the property of Portland cement. Physical & Chemical properties have been shown in Table 3.4 & 3.5 respectively.

Silica fume improves the durability of concrete in a wide variety of aggressive environments. The beneficial effects are seen at an early stage because silicafume reacts with calcium hydroxide almost as rapid as it is formed in the cement during hydration. The overall effect of removing calcium hydroxide, refining the pore structure and densifying the interfacial zone, is to reduce rebar corrosion,sulphate attack, acid attack, freeze thaw damage, Alkali-silica reaction, Efflorescence.

Table 3.4: Physical Properties of Silica Fume

Sr. No.	Property	Unit	Test result
1	Average Particle Size	m	1.5
2	Bulk density	gm/ltr	300 ± 30
3	Physical formation	-	Off-white powder
4	Specific Gravity	-	2.5
5	Brightness	-	80 ± 2
6	BET surface area	m^2/gm	15
7	Pozzolanic Reactivity	-	1050 gm $Ca(OH)_2$/gm

Table 3.5: Chemical Properties of Silica Fume

Sr. No.	Chemical Composition	% wt
1	$SiO_2 + Al_2O_3 + Fe_2O_3$	96.88
2	CaO	0.39
3	MgO	0.08
4	TiO_2	1.35
5	Na_2O	0.56
6	K_2O	0.06
7	Li_2O	NIL
8	Loss on Ignition	0.68

3.2.4 Fibres

Carbon steel hook end fibres are used in the present investigation. Physical properties of fibres are mentioned in Table 3.6. Pictorial view of steel fibres have been presented in Figure 3.1

Table 3.6: Properties of Steel Fiber

Type of Fibre	Length(mm)	Diameter(mm)	Density(kg/m^3)
Steel fibre	60	0.75	7850

Figure 3.1: Close View of Steel Fibres

3.2.5 Aggregate

Locally available 10 mm and 20 mm crushed aggregates have been used as coarse aggregates. Locally available river sand is used as fine aggregate for concrete. The aggregates are tested for properties in accordance with the IS standards. Tests for fine and coarse aggregates are conducted as per IS 2386[21] and IS 383[22], respectively. Physical properties and sieve analysis results of 20 mm aggregates, 10 mm aggregates and fine aggregate are presented in Table 3.7, 3.8 and 3.10, respectively.

Table 3.7: Gradation of Coarse Aggregate (20 mm)

Sieve Size	Mass Retained (gms)	% of Mass Retained	Cumulative % of Mass Retained	Cumulative % of Passing
80 mm	0	0	0	100
40 mm	0	0	0	100
20 mm	650	32.5	32.5	67.5
10 mm	1270	63.5	96	4
4.75 mm	80	4	100	0
2.36 mm	0	0	100	0
1.18 mm	0	0	100	0
600 micron	0	0	100	0
300 micron	0	0	100	0
150 micron	0	0	100	0
Total	2000	100	728.5	
Fineness modulus = 728.5/100 = 7.29				

Table 3.8: Gradation of Coarse Aggregate (10 mm)

Sieve Size	Mass Retained (gms)	% of Mass Retained	Cumulative % of Mass Retained	Cumulative % of Passing
80 mm	0	0	0	100
40 mm	0	0	0	100
20 mm	0	0	0	100
10 mm	63	6.3	6.3	93.7
4.75 mm	902	90.2	96.5	3.5
2.36 mm	35	3.5	100	0
1.18 mm	0	0	100	0
600 micron	0	0	100	0
300 micron	0	0	100	0
150 micron	0	0	100	0
Total	1000	100	602.8	
Fineness modulus = 602.8/100 =6.028				

With reference to Table 3.9 and IS 383-1970 (Table 4)[22], sand is under zone-II category.

Table 3.9: Gradation of Fine Aggregate

Sieve Size	Mass Retained (Grams)	% of Mass Retained	Cumulative % of Mass Retained	Cumulative % of Passing
80 mm	0.00	0.00	0.00	100.00
40 mm	0.00	0.00	0.00	100.00
20 mm	0.00	0.00	0.00	100.00
10 mm	3.50	0.35	0.35	99.65
4.75 mm	14.30	1.43	1.78	98.23
2.36 mm	31.30	3.13	4.90	95.10
1.18 mm	313.50	31.35	36.25	63.75
600 micron	207.00	20.70	56.95	43.05
300 micron	272.20	27.22	84.16	15.84
150 micron	110.50	11.05	95.22	4.78
Lower than 150	0.00	2.17	-	2.62
Total	1000	100	279.61	
Fineness Modulus = 279.61/100 = 2.79				

Table 3.10: Aggregate Properties

Material	Loose Bulk Density (kg/cu.mt.)	Compact Density (kg/cu.mt.)	Specific Gravity
Sand	1532	1671	2.57
Aggregate (10 mm)	1348	1509	2.73
Aggregate (20 mm)	1542	1601	2.73

3.2.6 Chemical Admixture

Superplasticizer has been used to achieve proper workability of control concrete. Auramix-300 has been used to improve workability and reduce W/C ratio of fresh concrete. Table 3.11 shows the chemical properties of Auramix-300.

Table 3.11: Properties of Chemical Admixture

Sr.No.	Parameter	Observation
1.	Physical state	Light yellow liquid
2.	Chemical name of the active ingredient	Polycarboxylic ether
3.	pH	6
4.	Chloride content	Nil

Marsh Cone Test

Determination of optimum dosage of superplasticizer plays a very important role in making durable and long-lasting concrete. This is done with the help of Marsh Cone test. In this experiment, the time taken for cement paste with different dosage of superplasticizer is measured. The super plasticizer selected is Fosroc Auramix-300. Table 3.12 presents the results of Marsh Cone Test. Super plasticizer dosage given in percentage is with respect to the weight of cement taken in the mix of cement paste. Figure 3.2 represents the results obtained from the test in graphical form. The optimum dosage is the amount when the curve almost becomes flat.

It is observed that the optimum dosage of superplasticizer is 0.7% by weight of cement as presented in Figure 3.2

Table 3.12: Mars Cone Test Results

Sr.no.	Superplasticizer Dosage %	Time (second)
1.	0.4	80
2.	0.5	60
3.	0.6	56
4.	0.7	50
5.	0.8	48
6.	0.9	46

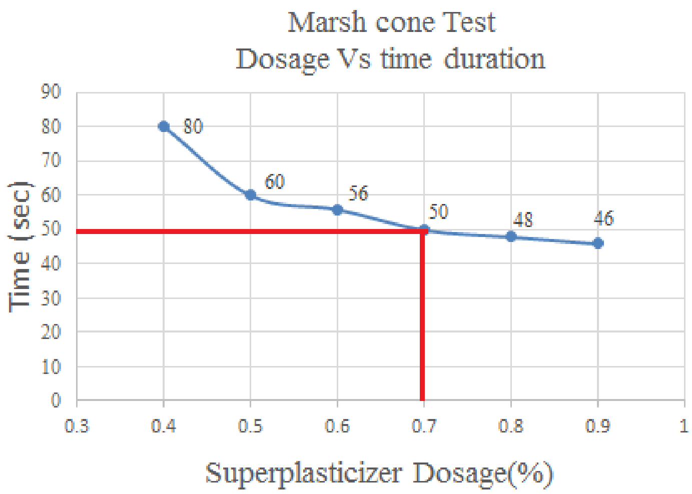


Figure 3.2: Marsh Cone Test

3.3 Concrete Mix Design

The details about mix design procedure conducted for Normal strength concrete(NSC-M25) & High strength concrete(HSC-M60) has been explained in this section.

3.3.1 Normal Strength Conrete - M25 Grade

Control concrete (CC) mix design is based on provisions of IS 10262
Step 1: As per IS: 10262,Table-2,[26]
For 20 mm size of aggregate,
Maximum water content = 186 litre (for 25 to 50 mm slump range)
Estimated water content for 100 mm slump = 186 + 6 $\times$ (186 $\div$ 100) = 197.16 litre.

As superplasticizer has been used, the water content can be reduced up 20 percent and above.
Based on trials with superplasticizer FOSROC Auramix-300 water content reduction of 17 percent has been achieved. Hence arrived water content = 197.16 $\times$ 0.83 = 164 litres

Step 2: Calculation of Cement Content.
Water Cement ratio = 0.45
Cement Content = 164/0.45 = 364.4 kg $\approx$ 365 kg

Step 3:
For max. aggregate size = 20 mm
Fine aggregate (FA) zone = 2 & w/c ratio = 0.5, (Table 3, IS-10262 [10])
Coarse Aggregate (CA) content =0.62
But for selected w/c ratio 0.45,
0.05 $\rightarrow$ 0.01
(0.5-0.45) $\rightarrow$ 0.01
CA= 0.62+0.01 = 0.63
FA=1-0.63 = 0.37

Chemical admixture is taken 0.7% of mass of cementitious material from Marsh cone test.

Step 4:
Volume of concrete = 1 m^3
Volume of cement = (365/3.18) $\times$ (1/1000) = 0.11 m^3
Volume of water = (164/1) $\times$ (1/1000) = 0.16 m^3
Volume of chemical admixture (@ 0.7% by mass of cementitious material) = (2.5/1.101) $\times$ (1/1000) = 0.0026 m^3
Volume of all aggregate = 1 - 0.13 - 0.16 - 0.002 = 0.725 m^3 Weight of CA = 0.725 $\times$ 0.63 $\times$ 2.73 $\times$ 1000 = 1218 kg
Weight of 20 mm aggregate = 1218 $\times$ 0.6 = 731 kg
Weight of 10 mm aggregate = 1218 $\times$ 0.4 = 487 kg
Weight of FA = 0.725 $\times$ 0.37 $\times$ 2.57 $\times$ 1000 = 689 kg
Mix proportions of M25(PCC) mix are presented in Table 3.13.

Table 3.13: M25 Mix Proportions

Cement	365	kg/m^3
Fine Aggregate	689	kg/m^3
Coarse Aggregate (20 mm)	731	kg/m^3
Coarse Aggregate (10 mm)	487	kg/m^3
Admixture	2.5	kg/m^3
Water	164	$lit./m^3$
W/C ratio	0.45	-
Steel Fibre(for M25-FRC) (1% of volume of concrete)	78.5	kg/m^3

Mix design Proportion of M25(Fibre reinforced concrete) mix are same as mentioned in Table 3.13. In addition, 1% of steel hooked fibres by volume of concrete have been incorporated in concrete mix. Steel fibres are having length of 60 mm & aspect ratio of 64.

$$Fibredosagefor1m^3 = \frac{Fiberdensity \times Dosageoffibre(\%) \times VolumeofConcrete(m^3)}{(100)} \tag{3.2}$$

$$Fibredosagefor1m^3 = \frac{7850(kg/m^3) \times 1\%ofsteelfibre \times VolumeofConcrete(m^3)}{(100)} = 78.5kg \tag{3.3}$$

3.3.2 High strength Concrete - M60 Grade

Step 1:
Calculation of Cement Content and Pozzolanic Material

Water/Binder ratio = 0.28 (Based on ACI 211.4R-93)[31]

Taken total Binder Content(cement+fly ash+silicafume)=560 kg
Binder includes cement-53 grade,fly ash and silica fume.

Cement has been replaced by 8% of silica fume & 15% of fly ash by weight of total binder content. Fly ash content= 560 $\times$ 0.15 =85 kg
Silica fume content= 560 $\times$ 0.08 =45 kg

Cement Content= 570 - 85 - 45 = 440 kg <450 kg (max. allowable cement content)

Water Content = binder content $\times$ w/b ratio = 570 $\times$ 0.28 = 160 litres

Above calculated water includes reduction corrections due to superplasticizer.

Step 2:

Table 3.14: M60 Mix Proportions

Material	Content	Unit
Cement	440	kg/m^3
Fly ash (15% of binder)	85	kg/m^3
Silica Fume (8% of binder)	45	kg/m^3
Fine Aggregate	704	kg/m^3
Coarse Aggregate (20 mm)	516	kg/m^3
Coarse Aggregate (10 mm)	516	kg/m^3
Admixture (PC based)	4	kg/m^3
Water	160	Lit./m^3
W/C ratio	0.28	-
Steel Fibre (for M60-FRC) (1% of volume of concrete)	78.5	kg/m^3

Max size of coarse aggregate=20 mm
Fine aggregate (FA) zone = 2 & w/c ratio = 0.28, (Table 3, IS-10262 [10])
Coarse Aggregate (CA) content = 0.62
But for high strength concrete, to achieve dense micro-structure coarse aggregate has been taken as 58% of all aggregate Fine aggregate taken as 42% of all aggregate.

Step 3:
Volume of concrete = 1 m^3
Volume of cement =(440/3.18) × (1/1000) = 0.138 m^3
Volume of fly ash =(85/2.57) × (1/1000) = 0.032 m^3
Volume of silica fume =(45/2.5) × (1/1000) = 0.018 m^3
Volume of water = (160/1) × (1/1000) = 0.16 m^3

Volume of all aggregate = 1-0.138-0.032-0.018-0.16 = 0.652 m^3
Weight of CA = 0.652 × 0.58 × 2.73 × 1000 = 1032 kg
Weight of 20 mm aggregate = 1218 × 0.5 = 516 kg
Weight of 10 mm aggregate = 1218 × 0.5 = 516 kg
Weight of FA = 0.652 × 0.42 × 2.57 × 1000 = 704 kg

Mix design Proportion of M60(Fibre reinforced concrete) mix are same as mentioned in Table 3.14. In addition, 1% of steel hooked fibres by volume of concrete have been incorporated in concrete mix. Steel fibres are having length of 60 mm & aspect ratio of 64. 78.5 kg of steel fibres has been added for 1m^3 concrete volume.

3.4 RC Column Design

RC column is designed as per provision of IS 456[30]. The RC column design is conducted on the basis of M25 control concrete design. Reinforcement in category concrete mixes is same. Detailing drawing has been prepared for columns on the basis of design.

3.4.1 RC Column Design for M25 Grade Concrete

The calculations for RC column design for M25 grade control concrete are presented below:

Breadth B	= 150 mm
Depth D	= 150 mm
Length l	= 1.0 m
P_u	= 250 kN
Grade of concrete	= 25 N/mm^2
Grade of steel	= 415 N/mm^2
Cover	= 25 mm
Type of column	= Unbraced
k	= 0.65

Check for Slenderness $l_{eff} = 0.65 \times 1.0 = 0.65$ m
$l_x/D_x = 650/150 = 4.33 < 12$
$l_y/D_y = 650/150 = 4.33 < 12$
The column shall be designed as Short column
Minimum eccentricity IS:456-2000[31]
$e_{x\ min} = 650/500 + 150/30 = 6.3$ mm
$e_{y\ min} = 650/500 + 150/30 = 6.3$ mm

Moment due to minimum eccentricity
$M_{ux} = 250 \times 20/1000 = 5$ kN.m
$M_{uy} = 250 \times 20/1000 = 5$ kN.m
Avg. $M_u = \sqrt{(5^2 + 5^2)} = 7.07$ kN.m

To Prefer interaction diagram from Sp:16[32]
Diameter of stirrups = 8 mm
d' = 25 mm
P_u/f_{ck} bD = $250 \times 10^3/(25 \times 150 \times 150) = 0.44$
M_u/f_{ck} bD^2 = $7.07 \times 10^6/(25 \times 150 \times 150^2) = 0.084$
d'/D = 0.17
from SP:16 Chart 46, we get
p/f$_{ck}$ = 0.07
p = $0.07 \times 25 = 1.75\%$
A_{st} required $= 1.75 \times 150 \times 150/100 = 393.75$ mm^2

As per Cl:26.5.3.2 c(2) / IS 456-2000[30]
The diameter of the tie shall be

- maximum diameter of longitudinal bar $\times$ 1/4 or
- 6mm, whichever is maximum

Diameter of ties = 8 mm

As per Cl:26.5.3.2 / IS 456-2000[30]
Spacing of ties:

Provide 8 mm ties at spacing of 150 mm c/c

Check for moment capacities
$P_{provided}$ = 2.01
P_u/f_{ck} bD = 0.045
p/f_{ck} provided = 0.07
d'/D_x = 0.17
d'/D_y = 0.17
from SP:16 graphs, we get
M_{ux1}/f_{ck} bD^2 = 0.09
M_{uy1}/f_{ck} bD^2= 0.09
i.e., M_{ux1} = 0.09 × 25 × 150 × 150^2 = 7.59 kN.m
M_{uy1} = 0.09 × 25 × 150 × 150^2 = 7.59 kN.m
$M_{ux1} > M_{ux}$
$M_{uy1} > M_{uy}$

P_{uz} = [(0.45 × 25 × 150 × 150)+(0.75 × 415 × 452)]= 393.81 kN
P_u/P_{uz} = 250/393.81 = 0.64 < 0.80
α_n = 1.73
Check
$(5/7.59)^{1.73} + (5/7.59)^{1.73} = 0.971 <1$
Section safe as short column.

Reinforcement detailing of column cast with M25 grade concrete has been presented in Figure 3.3.

3.4.2 RC Column Design for M60 Grade Concrete

The calculations for RC column design for M60 grade concrete are presented below:

Breadth B	= 150 mm
Depth D	= 150 mm
Length l	= 1.0 m
P_u	= 500 kN
Grade of concrete	= 60 N/mm^2
Grade of steel	= 415 N/mm^2
Cover	= 25 mm
Type of column	= Unbraced
k	= 0.65

Check for Slenderness l_{eff} = 0.65 × 1.0 = 0.65 m
l_x/D_x = 650/150 = 4.33 < 12
l_y/D_y = 650/150 = 4.33 < 12
The column shall be designed as Short column
Minimum eccentricity IS:456-2000[30]
$e_{x\ min}$ = 650/500+150/30 = 6.3 mm
$e_{y\ min}$ = 650/500+150/30 = 6.3 mm

Moment due to minimum eccentricity
M_{ux} = 500 × 20/1000 = 10 kN.m
M_{uy} = 500 × 20/1000 = 10 kN.m
Avg. $M_u = \sqrt{(5^2 + 5^2)}$ = 14.14 kN.m

To Prefer interaction diagram from Sp:16[32]
Diameter of stirrups = 8 mm
d' = 25 mm
P_u/f_{ck} bD = 500 × 10^3/(60 × 150 × 150) = 0.37
M_u/f_{ck} bD^2 = 14.14 × 10^6/(25 × 150 × 150^2) = 0.07
d'/D = 0.17
from SP:16 Chart 46, we get
p/f_{ck} = 0.03
p = 0.07 × 25 = 1.8%
A_{st} required =1.8 × 150 × 150/100 = 405 mm^2

As per Cl:26.5.3.2 c(2) / IS 456-2000[30]
The diameter of the tie shall be

- maximum diameter of longitudinal bar × 1/4 or
- 6mm, whichever is maximum

Diameter of ties = 8 mm

As per Cl:26.5.3.2 / IS 456-2000[30]
Spacing of ties:
Provide 8 mm ties at spacing of 150 mm c/c

Check for moment capacities
$P_{provided}$ = 2.01
P_u/f_{ck} bD = 0.037
p/f_{ck} provided = 0.033
d'/D_x = 0.17
d'/D_y = 0.17
from SP:16 graphs, we get
M_{ux1}/f_{ck} bD^2 = 0.065
M_{uy1}/f_{ck} bD^2= 0.065
i.e., M_{ux1} = 0.075 × 60 × 150 × 150^2 = 15.18 kN.m
M_{uy1} = 0.075 × 25 × 150 × 150^2 = 15.18 kN.m
$M_{ux1} > M_{ux}$
$M_{uy1} > M_{uy}$

P_{uz} = [(0.45 × 25 × 150 × 150)+(0.75 × 415 × 452)]= 748 kN
P_u/P_{uz} = 500/748 = 0.66 < 0.80
α_n = 1.8
Check
$(10/15.18)^{1.8} + (10/15.18)^{1.8} = 0.95 < 1$

Section safe as short column.

Reinforcement detailing of column cast with M60 grade concrete has been presented in Figure 3.3.

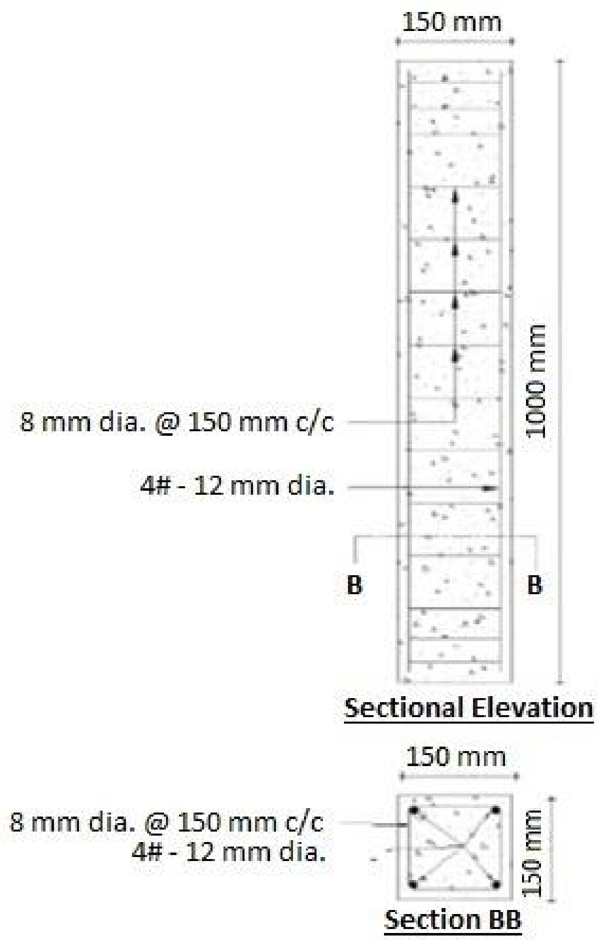


Figure 3.3: Reinforcement Detailing of Column

3.5 Manufacturing of Concrete

Weighing and batching process of all ingredients of concrete such as cement, fine aggregate and coarse aggregate i.e. 10 mm and 20 mm, water and superplasticizer is done with required accuracy before starting the mixing process. First all coarse aggregates and fine aggregates are added in drum mixer and mixing is continued the 20 to 25 second to make mix consistent. The cement is added in the mixer and mixing drum is allowed to rotation continuously. At time of mixing, water and admixture are required to be added gradually in the mixer drum. The machine is rotated till the uniform mix is achieved. The needle vibrator is used for proper compaction of Concrete for RC elements. Concrete drum mixture has been presented in Figure 3.4.

Figure 3.4: Concrete Drum Mixture

3.6 Casting of RC Column

Sixteen RC columns have been cast for experimental study. Available form-work in yard has been used for the casting of RC columns. The formwork used for the casting and the dimensions of the formwork are as shown in Figure 3.5. As it was R.C.C, it required application for oil inside the formwork before casting of each column specimen. The concrete cover of 25 mm size around all the sides are provided. After ensuring oil application, cover position of reinforcement cage was positioned in the form work, concrete of selected proportion was cast and poured inside the formwork. Needle vibrator was used for proper compaction of the concrete. After 24 hours of casting, formwork was removed and the specimens were kept 28 days for curing with gunny bags. Three cubes of each concrete mix also have been cast to measure compressive strength of concrete during casting of each batch of concrete.

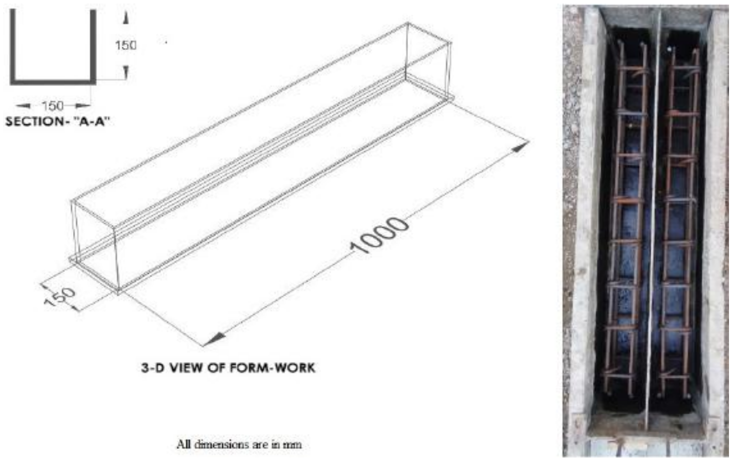


Figure 3.5: Steel Formwork with Reinforcement Cage

3.7 Testing Procedure of Concrete Elements Subjected to Elevated Temperatures

This section includes Basic information about Furnace, Time-temperature relationship & heating regimes to test concrete specimens at different elevated temperatures i.e. 300°C, 500°C, 700°C & 900°C.

3.7.1 Salient Features of Automatic Gas Fired Furnace

1. Automatic gas fired furnace is having upto 1000°C heating capacity which operates by means of LPG gas as a fuel.

2. Inside dimensions of the furnace is 2.77m×0.6×m0.45m as presented in Figure 3.6a.

3. Furnace is having 2 burners installed along with its length as presented in Figure 3.6b.

4. K-type of thermocouples have been inserted from top of furnace to measure furnace temperature provided by individual burners.

5. Terra-wool coating has been applied on interior face of furnace for heat insulation, which can resist upto 1600°C of temperature as presented in Figure 3.6c.

6. Furnace temperature display screen is as presented in Figure 3.6d.

7. Furnace works on principle of auto cut-off system i.e. after setting a target temperature by controller on panel board once, burners will spark, flame will ignite and temperature will increase inside furnace. At a time, when furnace temperature

(a) Furnace with Opening Hole

(b) Burners Connected to Blowers

(c) Inside View of Furnace

(d) Temperature Control Panel

Figure 3.6: Various Elements of Gas Fired Furnace

(a) Temperature Display Screen

Figure 3.7: Infrared Thermometer

reaches target temperature, burners will automatically cut-off and temperature inside furnace will gradually reduce, at a one point burners will automatically ignite again and this cycle will continue over again.

3.7.2 Salient Features of Infrared Thermometer (Laser Temperature Gun)

1. Infrared thermometer is having temperature measuring capacity upto 1200°C.
2. Laser beamer is required to be projected on a heated specimen to measure temperature.
3. Emissivity is required to be set as per material emissivity value. Emissivity value for concrete is 0.95 as presented in Figure 3.7.

3.7.3 Procedure for Concrete Specimens Subjected to Fire Exposure

1. Place concrete specimens (PCC or RCC) inside the furnace tunnel through openings.
2. Switch on the blowers. Set the required temperature on panel by temperature controller switches. Switch on the burners as presented in Figure 3.6d.
3. Temperature is required to be modified as per the Experimental time-temperature relationship at a certain periodical interval of time as presented in Table 3.15. Comparison of IS-3809 standard curve & experimental curve has represented is Figure 3.8.
4. For this particular study, duration of fire exposure at target temperature is taken as 60 minutes. Heating regime of furnace for testing specimens at different elevated temperature i.e. for 300°C, 500°C, 700°C & 900°C.
5. Rise in specimen temperature along with heating regime of furnace for different elevated temperatures have been measured with infrared thermometer for M25(PCC) grade specimens i.e. for 300°C, 500°C, 700°C & 900°C.
6. After 60 minutes of exposure duration at target temperature, specimens are allowed to air cool for 24 hours.
7. After allowing to air-cool at room temperature, Specimen will be proceed for destructive testing.

3.7.4 Representation of Experimental Time-Temperature Curve

Table 3.15 represents relationship of Experimental time-temperature curve along with standard IS:3809(1979)[15] time-temperature curve. Time-temperature relationship of IS:3809(1979) is based on $T\text{-}T_0$=345log(8t+1), where t=time in minutes, T_0=Initial temperature of furnace in °C & T = Furnace temperature in °C at time 't'. Experimental curve has been introduced as per available facility of gas fired furnace.

Table 3.15: Representation of Experimental Curve With IS:3809(1979)[15] Curve

Time (minutes)	Experimental Curve (°C)	IS:3809(1979)[15] (°C)
0	27	27
2.5	300	456
3.5	500	505
5	550	556
7.5	600	616
10	650	658
12.5	700	691
15	750	719
17.5	775	741
20	800	761
22.5	800	779
25	850	795
27.5	850	809
30	900	822
32.5	910	834
35	890	845
37.5	910	855
40	890	865
42.5	910	874
45	890	882
47.5	910	890
50	890	898
52.5	910	905
55	890	912
57.5	910	919
60	890	925
62.5	910	931
65	890	937
67.5	910	943
70	890	948
72.5	910	954
75	890	959
77.5	910	964
80	890	968
82.5	910	973
85	900	977

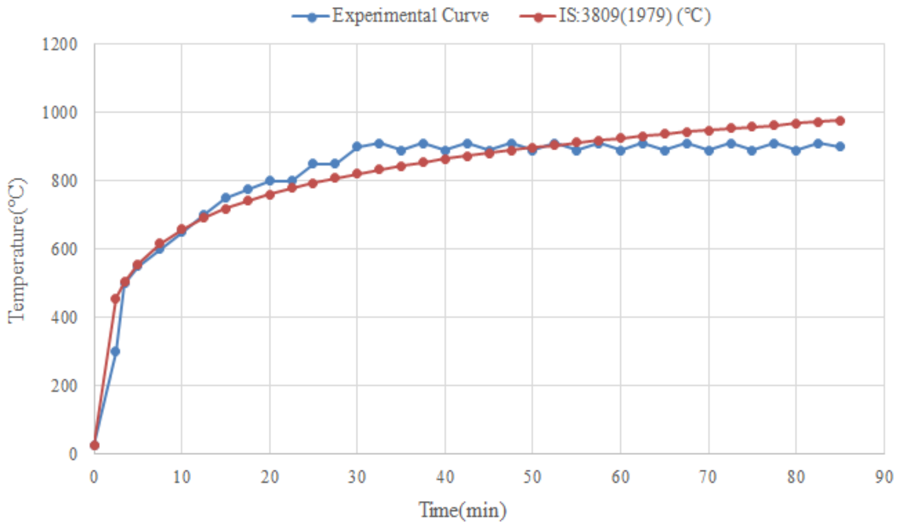


Figure 3.8: Graphical Representation of Experimental Curve & IS:3809(1979)[15] Recommended Curve

3.7.5 Heating Regimes for Different Elevated Temperatures

Heating regimens for different elevated temperatures have been represented along with rise in specimen temperatures have been represented in following figures. Exposure duration at target temperature has been taken for this particular study is 60 minutes. During testing of specimens(M25 Grade) at different elevated temperature, specimen temperature was measured using infrared thermometer. Measured Specimen temperatures at regular interval for different elevated temperatures 300°C, 500°C, 700°C & 900°C have been represented in Figure 3.9,3.10,3.11,3.12 respectively.

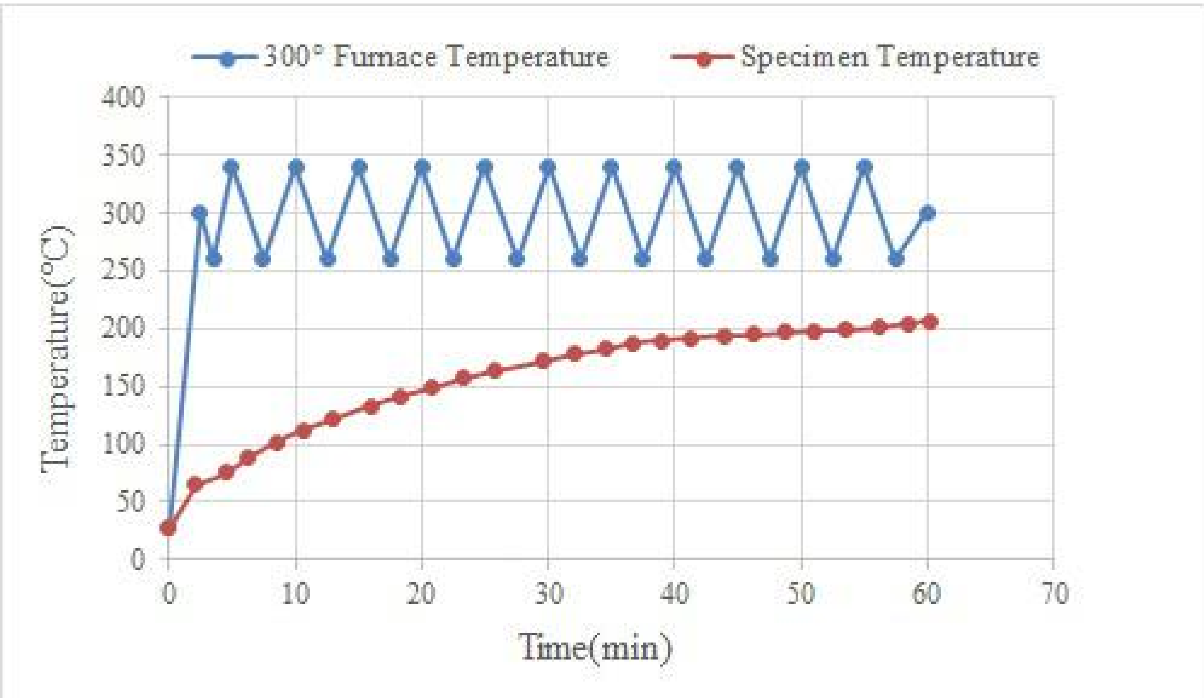


Figure 3.9: Rise in Specimens Temperature for 300°C Heating Regime

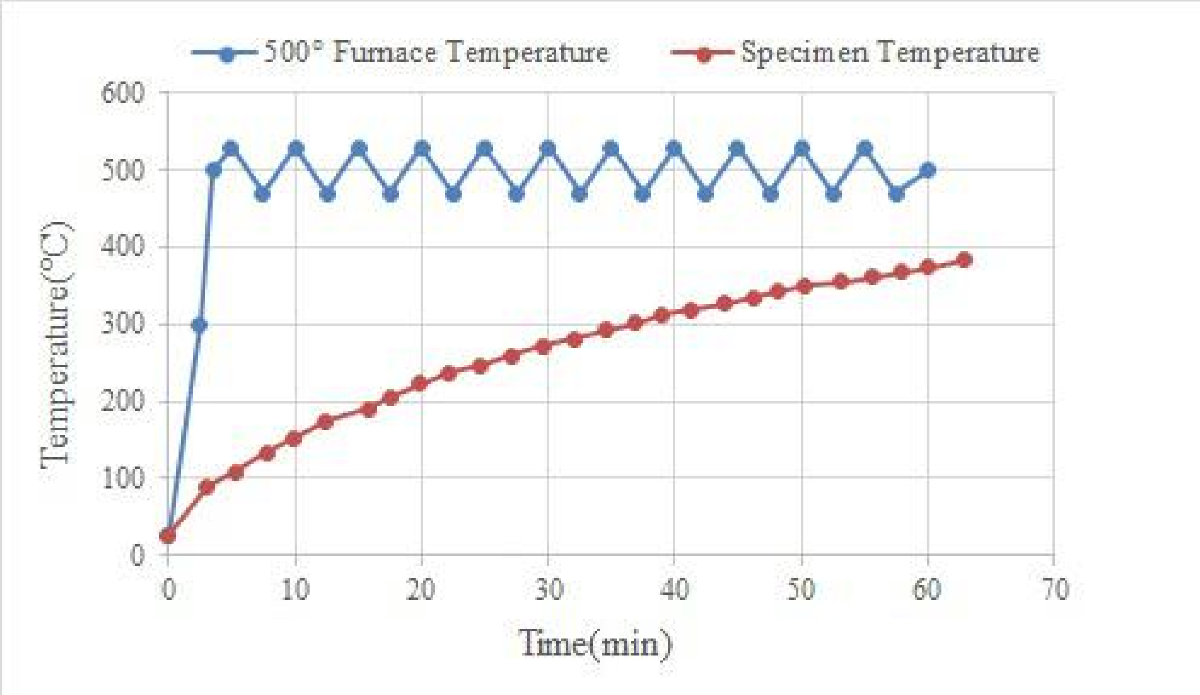


Figure 3.10: Rise in Specimens Temperature for 500°C Heating Regime

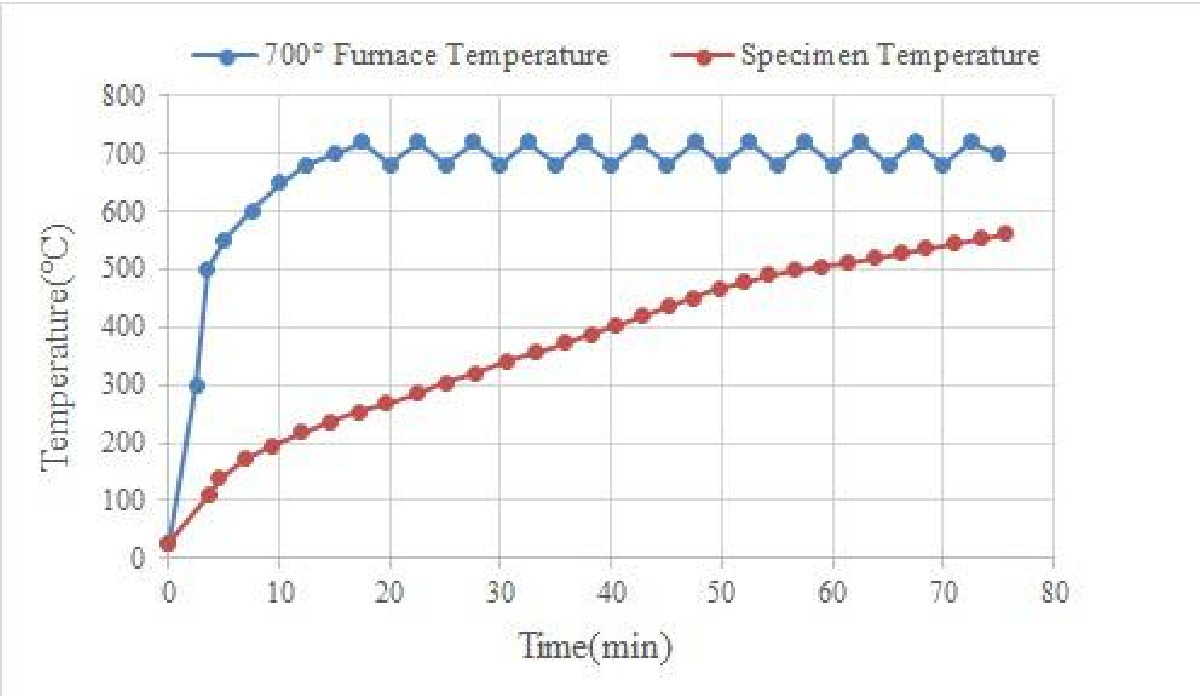


Figure 3.11: Rise in Specimens Temperature for 700°C Heating Regime

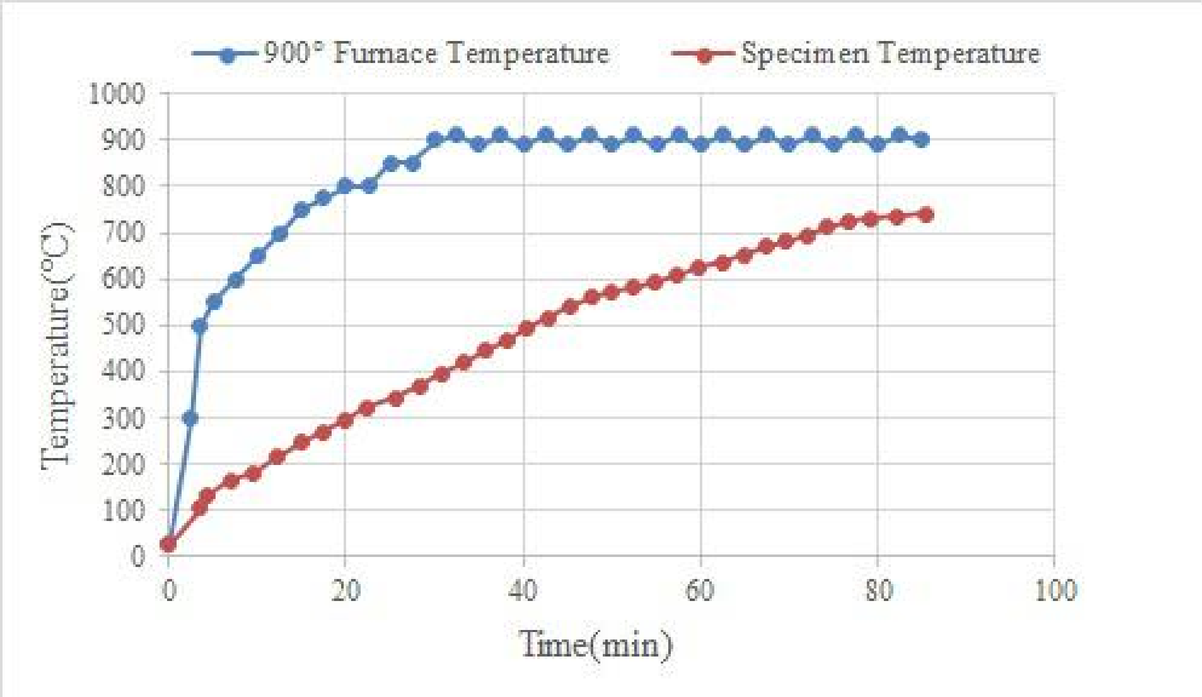


Figure 3.12: Rise in Specimens Temperature for 900°C Heating Regime

3.8 Procedure of Evaluation of Mechanical Properties of Plain Concrete Mixes

The discussion on different mechanical & durability properties such as compressive strength, flexural strength, split tensile strength, Modulus of elasticity & Bond strength on concrete mixes are covered in detailed in this section.

3.8.1 Compressive Strength

The compression testing machine of 2000 kN is used to evaluate compressive strength of both type of concrete mixes. For the compressive strength test, 150 mm × 150 mm × 150 mm cubes are tested in compression as per IS 516[19]. Equation of finding out compressive strength of the cube specimens is given below. Figure 3.13 shows test set up for finding compressive strength of cube.

$$Compressive\,Strength\,(N/mm^2) = \frac{P \times 10^3}{A} \tag{3.4}$$

P = Failure load of cube (kN)
A = Area of cube (150 × 150) (mm^2)

Figure 3.13: Plain Concrete Cube in Compression Testing Machine

3.8.2 Flexural Strength

The flexural strength is measured by performing flexural test on plain concrete specimens. The flexural testing machine of 100 kN is used to evaluate flexural strength of concrete specimens. For flexural strength test, beam of size 100 mm × 100 mm × 500 mm is cast in accordance with the test procedure given in IS 516[19]. Figure 3.14 shows set up for testing of beams. Figure 3.15 shows plain concrete specimen which is being tested in flexural testing machine.

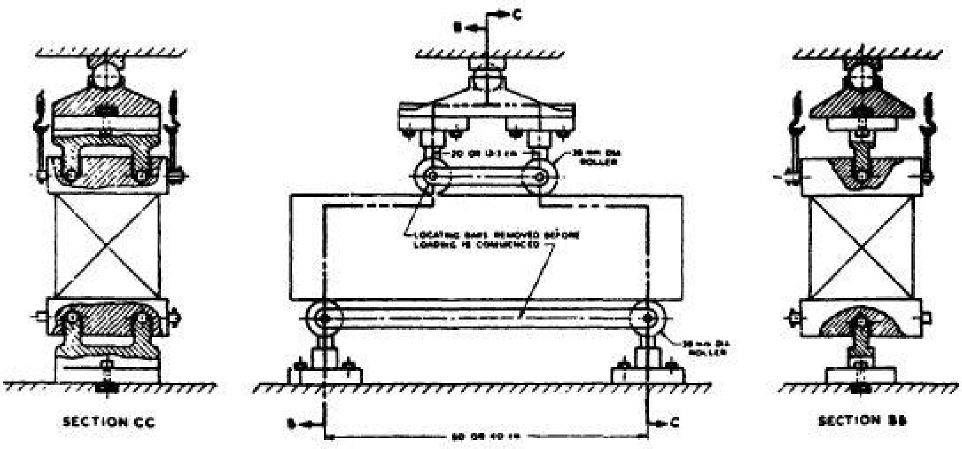


Figure 3.14: Flexure Test Setup

For evaluating the flexural strength of concrete beams, following eq. is used.

$$Flexural\,Strength\,(N/mm^2) = \frac{P \times L \times 10^3}{B \times d^2} \tag{3.5}$$

Where P = Failure load in kN
l = length between two support of concrete specimen in mm
b = width of the beam in mm
d = depth of the beam in mm

Figure 3.15: Plain Concrete Beam in Flexure Testing Machine

3.8.3 Split Tensile Strength

The compression testing machine of 2000 kN is used to evaluate split tensile strength of both type of concrete. Indirect method is used for finding the tensile strength of the concrete. For this test, cylinder of size 150 mm diameter × 300 mm height is tested in accordance with the test procedures given in IS: 5816[20].

Equation of finding out split tensile strength of the cylinder specimens is as given below.

$$Split\,Tensile\,Strength\,(N/mm^2) = \frac{2 \times P \times 10^3}{\pi \times L \times d} \tag{3.6}$$

P = Failure load of cylinder (kN)
L = Height of Specimen (300 mm)
d = Diameter of Specimen (150 mm)

Figure 3.16 shows the test set up for split tensile strength of concrete cylinder.

Figure 3.16: Test Setup for Split Tensile Strength of Concrete

3.8.4 Modulus of Elasticity

Modulus of Elasticity of concrete specimens has been determine with the help of extensometer. Cylinder specimen of 150 mm diameter and 300 mm height is used for evaluating the modulus of elasticity of concrete as per the test procedure given in IS 516[19]. The extensometers are fixed with the recording points at the same end. The load is applied continuously and without shock. Displacement is measured at equal load interval. Plot is drawn stress vs. strain from above results. Slope of the given plot by the tangent modulus proposes the modulus of elasticity of concrete specimen. Arrangement of concrete specimen for the said test is presented in Figure 3.17.

$$Stress = \frac{P \times 10^3}{A} \tag{3.7}$$

$$Strain = \frac{\delta l}{L} \tag{3.8}$$

$$MoE = \frac{Stress}{Strain} \tag{3.9}$$

Where,
P = Load (kN)
A = Area of loading surface = $\frac{\pi \times d^2}{4}$
δl = strain gauge reading at load P
L = Length between top and bottom screw mid point = 271 mm

Figure 3.17: Test Setup for Modulus of Elasticity of Concrete

3.8.5 Bond Strength

This test provides a standardized procedure for comparison of bond strength of all four mixes of concrete. Cube specimen of 150 mm with 12 mm diameter reinforcement has been used to evaluate the bond strength of concrete as per the test procedure given in IS 2770[24]. The test carried out is known as Pull-Out Test. In this Pull out test, the load is recorded at a relative slip of 0.002 mm at free end of the specimen. Bond strength is calculated by value obtained from failure load divided by the surface area of the embedded length of the bar.

The dial gauges used for measuring slip are having a least count of 0.0025mm. The testing machine has sufficient capacity to conduct the pull-out test. Dial gauges are attached with the specimen in such a way that movement of the reinforcing bar with respect to concrete is measured at both the loaded and unloaded ends of the bar. Figure 3.18 presents concrete specimen for pull out test. Figure 3.19 presents test setup for pull out test.

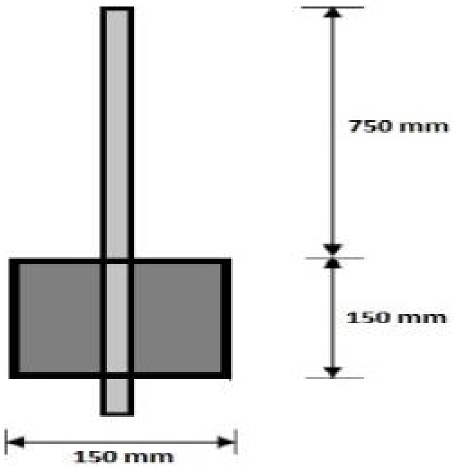


Figure 3.18: Bond Test Concrete Specimens

Three specimens of all four type of concrete mixes has been prepared and tested. The test specimens are mounted in universal testing machine in such a manner that the bar is pulled axially from the cube. The end of the bar at which the pull is applied is the one that projects from the face of the cube while it is being cast. The loading is applied to the reinforcing bar at the rate not greater than 230 N/min. The movements between the reinforcing bar and concrete cube, as indicated by the dial gauge is read at a sufficient number of intervals throughout the test to provide at least 15 readings by the time a slip of 0.25 mm occurs at the loaded end of the bar.

The loading is continued and the readings of the movement of reinforcement bar recorded at appropriate intervals until the yield point of the reinforcing bars is reached, the concrete cube has failed or minimum slippage of 2.5 mm occurred at the loaded end. Bond strength of both types of concrete is calculated by dividing the failure load at the slip specified, by the surface area of the embedded length of the bar. Average results of three elements are taken for the final result.

$$Bond\ Strength\ = \frac{P \times 10^3}{\pi \times L \times D} \tag{3.10}$$

Where,
P = Failure Load (kN)
L = Embedded Length of Reinforcement Bar
D = Diameter of Reinforcement Bar

Figure 3.19: Bond Strength Test of Reinforced Concrete

3.9 Testing of RC Columns

The discussion about RC column design, RC column loading condition, instrumentation & test setup of RC column are covered in this section.

3.9.1 Test Specimen

It has been planned to test columns for axial compressive load under the loading frame. Total 16 columns have been cast to study the axial load carrying capacity, deformation, stress-strain variation and cracks & failure patterns with same reinforcement. Total 16 columns have been divided into four categories as follows. Each category of column consists of 4 columns. Average results from two columns are to be considered for final results.

1. RC Column with M25
2. RC Column with M25(FRC)
3. RC Column with M60
4. RC Column with M60(FRC)

3.9.2 Test Setup for RC Column

Column specimens have been tested for axial compressive load under the loading frame. The load has been applied from the bottom through hydraulic jack of 1000 kN capacity. LVDT is attached to the column to measure the deformation of the column under the applied load. Strain gauge button are applied at the mid height to measure lateral strain. Test-setup with RC column under loading frame has been presented in Figure 3.20.

Following parameters have been evaluated during testing of each RC column:

1. Ultimate failure load
2. Load v/s displacement relationship
3. Axial stress v/s strain relationship
4. Failure modes and Crack patterns

3.9.3 Instrumentation for RC Column

To measure different parameters during experiments various type of instruments have been used as follows:-

1. Hydraulic Jack
2. LVDT
3. Mechanical Strain Gauges

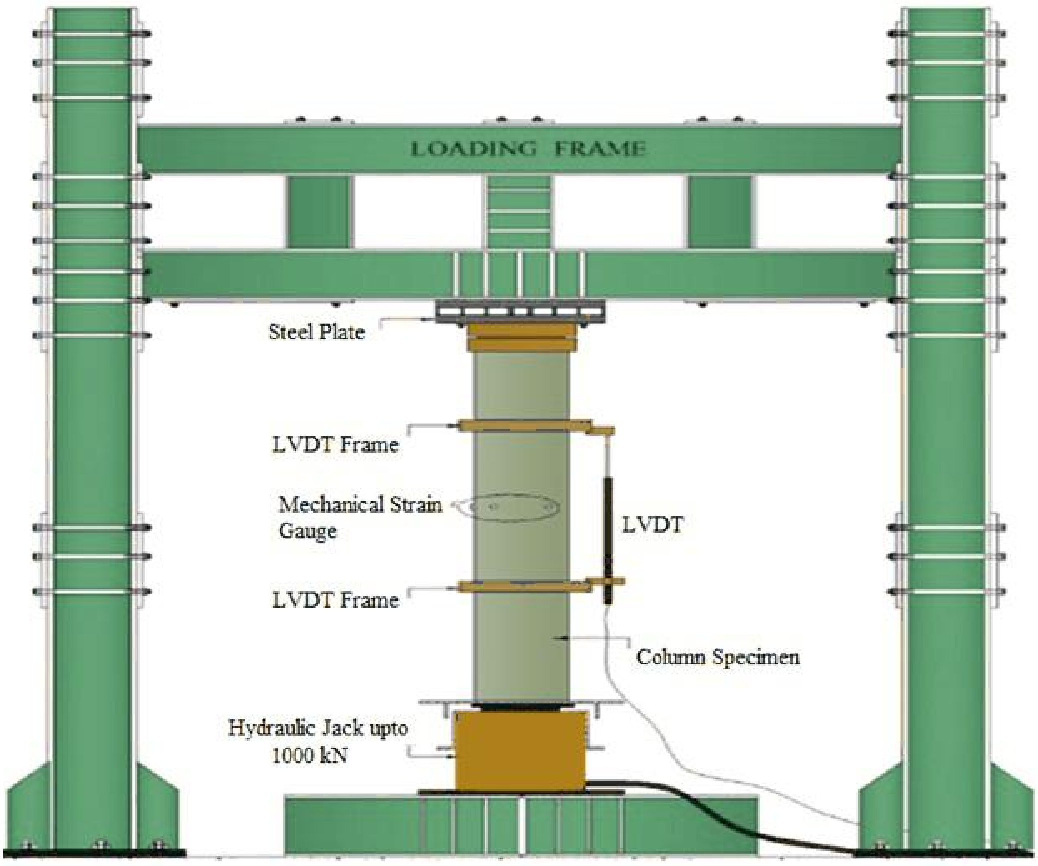


Figure 3.20: Test Set-up for RC Column

1) Hydraulic Jack

Hydraulic jack of capacity of 1000 kN has been used. Jack has been based on Pascals principle. Pressure is described, mathematically by a Force divided by Area. Therefore, if we assume two cylinders say one smaller and another larger has been connected together. As per Pascal's principle, force is applied to a smaller cylinder and the resultant pressure is achieved at the end of larger cylinder. Hydraulic jack which has been used for the application of loads is presented in Figure 3.21.

Figure 3.21: Hydraulic Jack

2) LVDT (Linear Variable Differential Transducer)

The vertical displacement of the specimen was measured with a linear variable differential Transducer (LVDT) with a travel of 50 mm which is mounted onto two aluminium frames that were located near the top and bottom of the specimen 600 mm apart, as shown in Figure 3.22.

Figure 3.22: LVDT Attached on Column Specimen

3) Mechanical Strain Gauges

Mechanical strain gauges which are also known as DEMEC (Demountable Mechanical) strain gauges. DEMEC gauges consist of an analogue dial gauge attached to an Invar bar. A fixed conical point is mounted at one end of bar. A moving conical point is mounted on a knife edge pivot at opposite end. Pivoting movement of second conical point is measured by dial gauge. A setting out bar is used to position pre-drilled stainless steel discs attached to column using a suitable adhesive. In this way, strain changes in column are converted into a change in reading on dial gauge. Instruments of mechanical strain gauge setup have been given in Figure 3.23.

Figure 3.23: Mechanical Strain Gauge

Chapter 4

Results & Discussion of Mechanical Properties of PCC Elements

4.1 General

In this chapter, behavior of M25(PCC), M25(FRC), M60(PCC) & M60(FRC) exposed to different elevated temperatures i.e. 300°C, 500°C, 700°C & 900°C have been discussed. Discussion in terms of physical changes & mechanical properties such as compressive strength, flexure strength, split tensile strength, modulus of elasticity & bond strength.

4.2 Physical Behavior of Different Concrete Mixes Exposed to Elevated Temperatures

In this section, physical properties of Normal strength concrete mixes & High strength concrete mixes exposed to different elevated temperatures have been discussed. Physical behavior such as spalling effect(type of spalling), crack pattern and weight loss have been discussed.

4.2.1 Behavior of Normal Strength Concrete(M25) Exposed to Different Elevated Temperatures

This section contains physical behaviour of Normal strength concrete exposed to different elevated temperatures. Comparison between M25(PCC) & M25(FRC) has been explained.

Figure 4.1 presents M25(PCC) elements after exposed to different elevated temperatures i.e. 300°C, 500°C, 700°C & 900°C.At 300°C there is no any significant physical change occurs as presented in Figure 4.1a. At 500°C there is a minor surface spalling is visible as presented in Figure 4.1b. At 700°C there is a edge spalling and micro-cracks are visible which is responsible for reduction in surface area which results into lesser load carrying capacity as presented in Figure 4.1c. At 900°C there is a major surface spalling & edge spalling is visible which is responsible for reduction in surface area which results into lesser load carrying capacity as presented in Figure 4.1d.

(a) Exposed to 300°C

(b) Exposed to 500°C

(c) Exposed to 700°C

(d) Exposed to 900°C

Figure 4.1: M25(PCC) Subjected to Different Elevated Temperatures

Figure 4.2 represents M25(FRC) elements after exposed to different elevated temperatures i.e. 300°C, 500°C, 700°C & 900°C. At 300°C there is no any significant physical change as presented in 4.2a.At 500°C there is a pothole surface spalling as presented in 4.2b. At 700°C & 900°C there is a major surface spalling as well as edge spalling & corner spalling and micro-cracks as well as macro-cracks are visible as presented in 4.2c. At 900°C there is a major surface spalling & edge spalling.Macro cracks are visible which is responsible for reduction in surface area which results into lesser load carrying capacity as presented in 4.2d. Spalling phenomena for 700°C & 900°C was higher, reason behind is at these temperatures steel fibres dispersed along outer surface gets volumetric change which debonds from concrete and concrete gets spalled.

(a) Exposed to 300°C

(b) Exposed to 500°C

(c) Exposed to 700°C

(d) Exposed to 900°C

Figure 4.2: M25(FRC) Subjected to Different Elevated Temperatures

(a) M25(PCC)

(b) M25(FRC)

Figure 4.3: Compressive Failure Pattern M25(PCC) & M25(FRC) after Exposed to 900°C

Figure 4.3 presents failure pattern of M25(PCC) & M25(FRC) in compression, after exposed to 900°C. It is clearly visible that M25(PCC) element gets explosive brittle failure while M25(FRC) element gets ductile failure.

Table 4.1: % Weight Loss in M25(FRC) Exposed to Different Elevated Temperatures

M25(FRC)				
Temperature(°C)	Condition	Specimen	Mass (kg)	% weight loss
300°C	Before	1	8.132	0.147
		2	8.175	
		3	8.200	
		Avg.	8.169	
	After	1	8.120	
		2	8.161	
		3	8.190	
		Avg.	8.157	
500°C	Before	1	8.165	0.262
		2	8.174	
		3	8.128	
		Avg.	8.156	
	After	1	8.140	
		2	8.156	
		3	8.107	
		Avg.	8.134	
700°C	Before	1	8.145	0.408
		2	8.220	
		3	8.130	
		Avg.	8.165	
	After	1	8.110	
		2	8.190	
		3	8.095	
		Avg.	8.132	
900°C	Before	1	8.150	0.689
		2	8.115	
		3	8.130	
		Avg.	8.132	
	after	1	8.075	
		2	8.082	
		3	8.070	
		Avg.	8.076	

Table 4.1 gives percentage weight loss of M25(FRC) cube elements after exposed to different elevated temperatures. Weight loss occurs due to spalling effect. For each range of temperature weight loss of 3 specimens have been taken and average value of weight loss of 3 specimens have been taken as final percentage weight loss.

4.2.2 Behavior of High Strength Concrete(M60) Exposed to Different Elevated Temperatures

(a) Exposed to 300°C

(b) Exposed to 500°C

(c) Exposed to 700°C

(d) Exposed to 900°C

Figure 4.4: M60(PCC) Subjected to Different Elevated Temperatures

Figure 4.4 represents M60(PCC) elements after exposed to different elevated temperatures i.e. 300°C, 500°C, 700°C & 900°C. Even at 300°C high strength concrete deteriorates which is not commonly visible in normal strength concrete as presented in Figure 4.4a. As temperature rises as represented in Figure 4.4b,4.4c,4.4d.concrete gets major volumetric changes. At extreme temperatures heavy corner spalling is visible. Reason behind severe spalling in high strength concrete is, as high strength concrete is having dense micro structure with higher density. At extreme temperature free water evaporates and tries to escape out from the concrete but due to dense micro-structure these is no proper gateway to escape which generates surface tension on surfaces of concrete elements results into heavy spalling.

(a) Exposed to 300°C

(b) Exposed to 500°C

(c) Exposed to 700°C

(d) Exposed to 900°C

Figure 4.5: M60(FRC) Subjected to Different Elevated Temperatures

Figure 4.5 presents M60(FRC) elements after exposed to different elevated temperatures i.e. 300°C, 500°C, 700°C & 900°C. As in case of M60(PCC) this mix too shows minor spalling effect at 300°C as presented in Figure 4.5a. As temperature rises as presented in Figure 4.5b,4.5c,4.5d concrete gets major volumetric changes. Corner spalling for M60(FRC) is lesser as compared to M60(PCC) because of fibre incorporation.Reason for heavy spalling at extreme temperature is same as for M25(FRC) mix, which is at elevated temperatures steel fibres are under influence of volumetric change which debonds concrete from surface of steel fibre.

Destructive tests have been carried out after 24 hours of cooling period. Destructive test to find out compressive strength of each mix after exposed to 900°C has been presented in Figure 4.6 Images represents that during destructive testing, PCC elements gets explosive failure and majority portion of specimen gets dismantled from the core concrete. Where in case of FRC elements, sudden explosive failure is not there. Fibres spread through out volume absorbs energy and arrest the crack propagation.

Table 4.2: % Weight Loss in M60(PCC) & M60(FRC) Exposed to Different Elevated Temperatures

			M60(PCC)		M60(FRC)	
Temperature	Condition	Specimen	Mass	% Weight loss	Mass	% Weight loss
300°C	Before	1	8.552	0.160	8.581	0.152
		2	8.575		8.575	
		3	8.557		8.566	
		Avg.	8.561		8.574	
	After	1	8.540		8.568	
		2	8.561		8.563	
		3	8.542		8.552	
		Avg.	8.548		8.561	
500°C	Before	1	8.541	0.378	8.578	0.300
		2	8.552		8.568	
		3	8.562		8.562	
		Avg.	8.552		8.569	
	After	1	8.515		8.553	
		2	8.519		8.547	
		3	8.524		8.531	
		Avg.	8.519		8.544	
700°C	Before	1	8.547	0.713	8.591	0.664
		2	8.558		8.572	
		3	8.564		8.581	
		Avg.	8.556		8.581	
	After	1	8.482		8.530	
		2	8.490		8.525	
		3	8.514		8.518	
		Avg.	8.495		8.524	
900°C	Before	1	8.551	1.008	8.56	0.782
		2	8.570		8.571	
		3	8.568		8.576	
		Avg.	8.563		8.569	
	After	1	8.462		8.480	
		2	8.481		8.511	
		3	8.487		8.515	
		Avg.	8.477		8.502	

Table 4.2 represents percentage weight loss for M60(PCC) & M60(FRC) cube specimens after exposed to different elevated temperatures. Results shows that percentage weight loss in case of M60(PCC) elements is higher as compared to M60(FRC). Incorporation of steel fibres strengthen bond between aggregates which results in lesser spalling.

As discussed earlier High strength concrete is prone to damage at elevated temperature as compared to normal strength concrete. Concrete brittleness increases with increase in temperature. Fig.4.6 represents destructive failure pattern of M60(PCC) heated at different elevated temperatures for compressive strength. Visual inspection represents that

(a) M60(PCC)

(b) M60(FRC)

Figure 4.6: Failure Pattern of M60(PCC) & M60(FRC) after Exposed to 900°C

(a) Exposed to 300°C

(b) Exposed to 500°C

(c) Exposed to 700°C

(d) Exposed to 900°C

Figure 4.7: M60(PCC) Destructive Failure Pattern Exposed to Different Elevated Temperature

as concrete is heated at elevated temperatures, failure pattern gets sudden and explosive in nature due to brittleness.

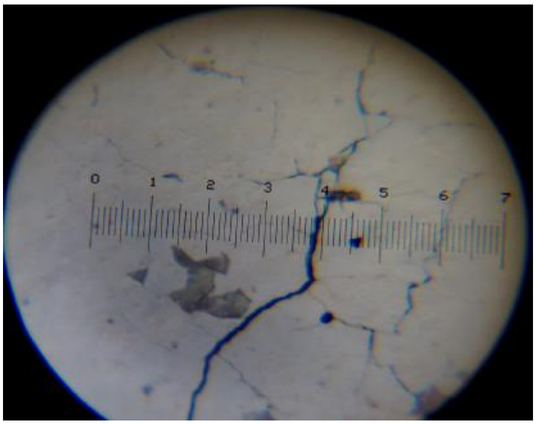
Figure 4.7 represents destructive failure pattern of M60(PCC) cube elements under compression. Figure shows that from exposure of 300°C to 900°C, M60(PCC) elements exhibits brittle explosive failure.

Crack Width Measurements

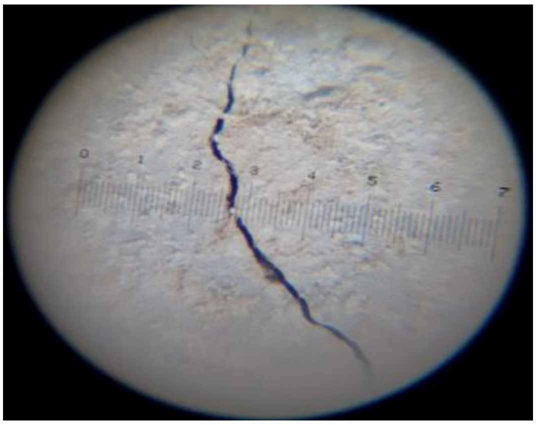
Figure 4.8: Pictorial View of Microscope

Crack measurement have been observed with help of microscope with least count of 0.1 mm as presented in Figure 4.8.

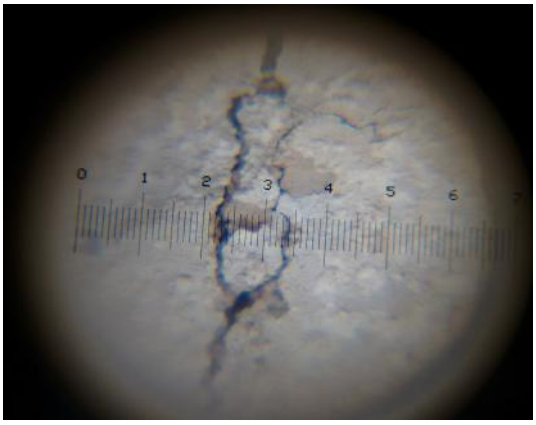
Cracks have been observed for elements produced from M60 grade of concrete after fire exposure. For 300°C & 500°C temperature micro cracks have been observed. For 700°C & 900°C micro cracks as well as macro cracks have been observed. Figure 4.9a denotes crack pattern of M60(PCC) exposed to 500°C. Figure 4.9b denotes crack pattern of M60(FRC) exposed to 700°C. Figure 4.9c denotes crack pattern of M60(FR) exposed to 900°C. Figure 4.9d denotes crack pattern of M60(PCC) exposed to 900°C.



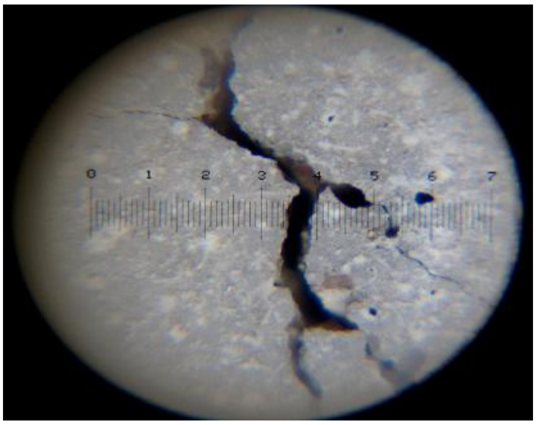
(a) 0.1mm at 500°C (M60 PCC-Cube)



(b) 0.15mm at 700°C (M60 FRC-Cylinder)



(c) 0.2mm at 900°C (M60 FRC-Cylinder)



(d) 0.5mm at 900°C (M60 PCC-Cube)

Figure 4.9: Crack Patterns in Different Mixes Exposed to Elevated Temperatures

4.3 Mechanical Properties

Discussion of the results of mechanical properties for Normal strength concrete & High strength concrete mixes after exposed to different elevated temperatures have been discussed. Mechanical properties includes compressive strength, split tensile strength, flexural strength, modulus of elasticity & bond strength.

4.3.1 Compressive Strength

The compressive strength of all 4 concrete mixes have been tested at the curing age of 28 days and presented in Table 4.3. Compressive strength of unheated specimens have been kept as base value for comparison purpose. Average values of 3 specimens have been taken as final result. Cube specimens of all 4 mixes were subjected to different elevated temperature for 60 minutes as per experimental time-temperature curve. Tested values of compressive strength have been compared with that of unheated specimens.

Table 4.3: Residual Compressive Strength of All Mixes Exposed to Different Elevated Temperatures

Temp. (°C)	M25(PCC)		M25(FRC)		M60(PCC)		M60(FRC)	
	Comp. Stren. (MPa)	Avg. (MPa)	Comp. Stren. (MPa)	Avg. (Mpa)	Comp. Stren. (MPa)	Avg. (MPa)	Comp. Stren. (MPa)	Avg (MPa)
Unheated	32.50	32.37	32.40	32.73	63.50	63.17	65.60	64.67
	32.70		33.80		61.70		64.80	
	31.90		32.00		64.30		63.60	
300°C	30.20	29.60	32.80	32.07	56.00	57.73	60.10	59.47
	29.80		31.50		59.60		59.60	
	28.80		31.90		57.60		58.70	
500°C	26.80	26.70	28.30	29.40	49.40	49.67	54.30	54.30
	27.30		30.20		51.00		53.50	
	26.00		29.70		48.60		55.10	
700°C	21.00	21.17	26.50	27.33	41.80	39.93	47.60	48.40
	20.40		27.40		40.40		49.60	
	22.10		28.10		37.60		48.00	
900°C	18.40	19.30	25.40	23.97	25.30	25.43	38.50	39.87
	19.20		23.80		26.20		39.70	
	20.30		22.70		24.80		41.40	

Percentage loss in compressive strength for M25(PCC) & M25(FRC) mixes have been observed as 8.55%, 2.04% for 300°C, 17.51%, 10.18% for 500°C, 34.60%, 16.50% for 700°C & 40.37%, 26.78% for 900°C. Results indicates that at each extreme temperatures(300°C, 500°C, 700°C, 900°C) plain cement concrete(PCC) suffers more in terms of spalling and compressive strength loss compared to fibre reinforced concrete(FRC) mix. Incorporation of steel fibre in normal strength concrete reduces loss in compressive strength at each elevated temperatures.

Percentage loss in compressive strength for M60(PCC) & M60(FRC) mixes have been observed as 8.60% & 8.04% for 300°C, 21.37% & 16.03% for 500°C, 36.78% & 25.15%

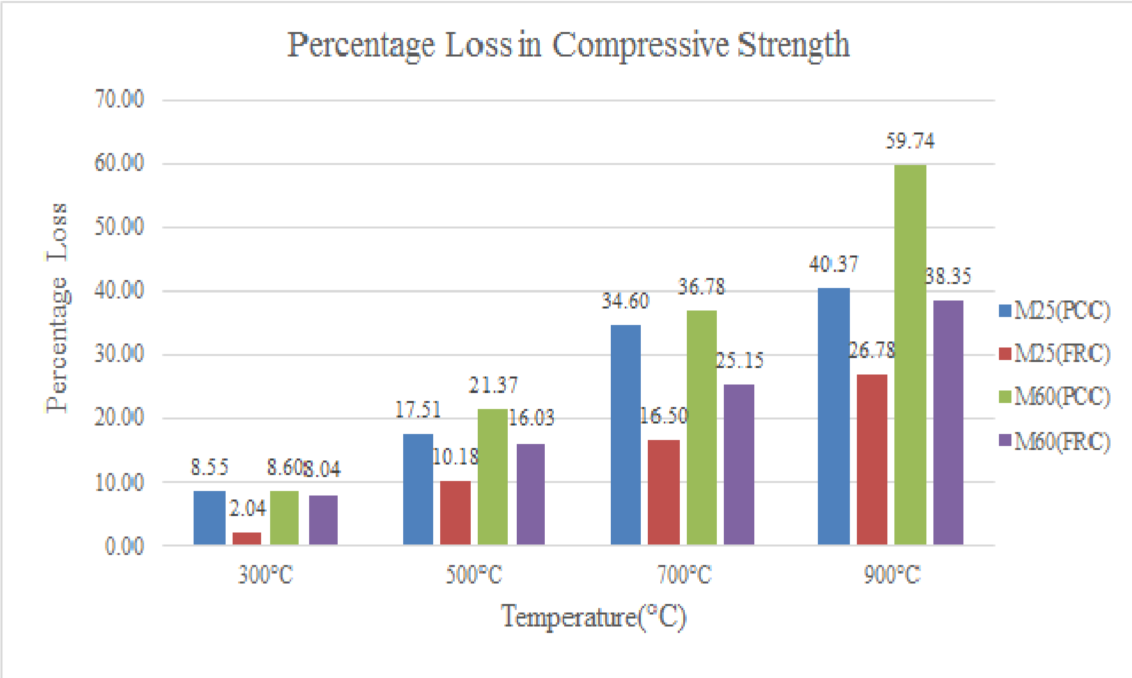


Figure 4.10: % Loss in Compressive Strength at Different Elevated Temperatures as Compared to Unheated Specimens

for 700°C & 59.74% & 38.35% for 900°C.Results indicates that for 300°C M60(PCC) & M60(FRC) is nearly equal but as temperature increases (500°C, 700°C, 900°C) compressive strength loss in M60(PCC) is higher as compared to M60(FRC). Reason behind is M60(PCC) exhibits higher spalling especially corner spalling & edge spalling which is responsible for reduce surface area which results in lesser compressive strength.

Figure 4.10 presents that as temperature increase, both Normal strength concrete(M25) & high strength concrete(M60) losses compressive strength. Result values indicates that, though during unheated condition steel fibres does not significantly affect the compressive strength, at higher temperature it plays major role to reduce compressive strength loss. Result shows that High strength concrete(HSC) is more vulnerable at elevated temperature compared to Normal strength concrete(NSC).

4.3.2 Split Tensile Strength

The split tensile strength of all 4 concrete mixes have been tested at the curing age of 28 days and presented in Table 4.4. Split tensile strength of unheated specimens have been kept as base value for comparison purpose. Average values of 3 specimens have been taken as final result. Specimens of all 4 mixes were subjected to different elevated temperature for 60 minutes as per experimental time-temperature curve. Tested values of split tensile strength have been compared with that of unheated specimens.

Table 4.4: Residual Split Tensile Strength of All Mixes Exposed to Different Elevated Temperatures

Temp. (°C)	M25(PCC)		M25(FRC)		M60(PCC)		M60(FRC)	
	Split Tensile Stren. (MPa)	Avg. (MPa)	Split Tensile Stren. (MPa)	Avg. (Mpa)	Split Tensile Stren. (MPa)	Avg. (MPa)	Split Tensile Stren. (MPa)	Avg. (MPa)
Unheated	3.50	3.28	4.40	4.33	6.21	6.18	8.44	8.44
	3.10		4.70		5.90		8.70	
	3.25		3.90		6.44		8.19	
300°C	3.10	3.08	4.20	4.03	5.82	5.72	8.15	7.98
	3.20		3.90		5.73		7.83	
	2.95		4.00		5.60		7.97	
500°C	2.90	2.85	3.80	3.93	5.35	5.01	7.56	7.25
	2.80		3.90		4.52		7.05	
	2.86		4.10		5.17		7.15	
700°C	2.21	2.47	3.30	3.57	3.65	3.97	6.47	6.34
	2.67		3.50		4.38		6.25	
	2.52		3.90		3.87		6.30	
900°C	1.69	1.69	2.60	3.00	2.68	2.82	5.62	5.38
	1.57		3.00		2.97		5.16	
	1.80		3.40		2.80		5.35	

Percentage loss in split strength for M25(PCC) & M25(FRC) mixes have been observed as 6.09%, 6.92% for 300°C, 13.10%, 9.23% for 500°C, 24.87%, 17.69% for 700°C & 48.63%, 30.77% for 900°C. In unheated condition Result indicates that at extreme temperatures M25(PCC) suffers more compare to M25(FRC). Plain concrete(M25-PCC) exhibits higher spalling especially corner spalling & edge spalling which is responsible for reduction in surface area which results in lesser split tensile strength as compared to M25(FRC).

Percentage loss in split strength for M60(PCC) & M60(FRC) mixes have been observed as 7.55% & 5.45% for 300°C, 18.92% & 14.09% for 500°C, 35.85% & 24.91% for 700°C & 54.45% & 36.32% for 900°C. Result indicates that in unheated condition steel fibre incorporation increases split tensile strength. As temperature increases loss split tensile strength for M60(PCC) is higher as compared to M60(FRC).

Figure 4.11 presents that as temperature increase, both Normal strength concrete(M25) & high strength concrete(M60) displays reduction in split tensile strength. Result values indicates that, steel fibre incorporation enhances split tensile strength during unheated

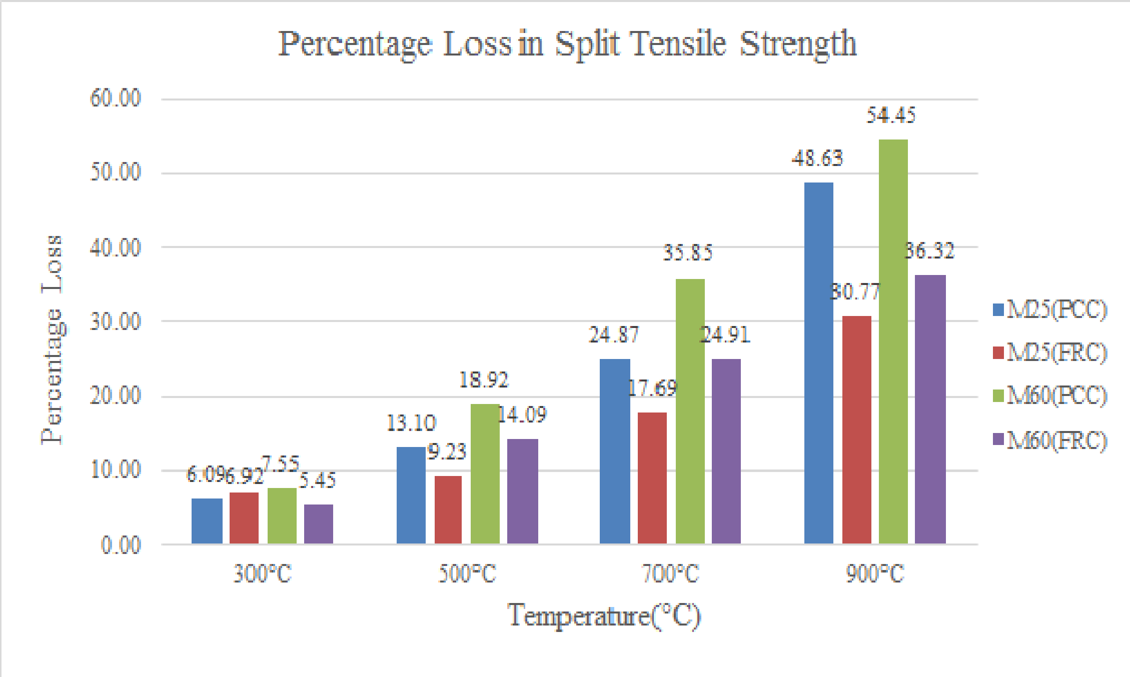


Figure 4.11: % Loss in Split Tensile Strength at Different Elevated Temperatures as Compared to Unheated Specimens

condition for Normal strength concrete as well as High strength concrete. For 300°C exposure, avg. strength loss of all 4 mixes are nearly equal. At extreme temperature FRC(M25 & M60) performs better compare to PCC(M25 & M60) respectively. Steel fibres exhibits ductility to specimens which avoids sudden brittle failure of cylinder specimens, which is common phenomena in PCC elements.

4.3.3 Flexural Strength

The Flexural strength of all 4 concrete mixes have been tested at the curing age of 28 days and presented in Table 4.5. Flexural strength of unheated specimens have been kept as base value for comparison purpose. Average values of 3 specimens have been taken as final result. Beam Specimens of all 4 mixes were subjected to different elevated temperature for 60 minutes as per experimental time-temperature curve. Tested values of flexural strength have been compared with that of unheated specimens.

Table 4.5: Residual Flexural Strength of All Mixes Exposed to Different Elevated Temperatures

Temp. (°C)	M25(PCC)		M25(FRC)		M60(PCC)		M60(FRC)	
	Flex. Stren. (MPa)	Avg. (MPa)	Flex. Stren. (MPa)	Avg. (Mpa)	Flex. Stren. (MPa)	Avg. (MPa)	Flex. Stren. (MPa)	Avg. (MPa)
Unheated	4.30	4.22	7.61	7.58	6.90	6.82	10.86	10.56
	4.10		7.89		6.75		10.60	
	4.25		7.23		6.80		10.21	
300°C	3.90	3.92	7.27	7.19	6.15	6.11	10.31	9.91
	4.00		6.98		6.23		9.89	
	3.85		7.32		5.95		9.54	
500°C	3.86	3.74	7.45	6.75	5.42	5.02	8.66	8.67
	3.70		6.90		4.68		8.96	
	3.65		5.89		4.96		8.40	
700°C	3.36	3.30	6.67	6.29	3.74	3.65	7.26	7.50
	3.24		5.97		3.60		7.80	
	3.29		6.23		3.60		7.45	
900°C	1.97	2.36	5.15	5.23	1.74	1.62	5.94	5.85
	2.25		4.90		1.26		6.15	
	2.87		5.64		1.86		5.45	

Percentage loss in flexural strength for M25(PCC) & M25(FRC) mixes have been observed as 7.11%, 5.10% for 300°C, 11.3%, 10.35% for 500°C, 21.82%, 16.98% for 700°C & 43.95%, 30.97% for 900°C as presented in Figure 4.12 FRC elements displays higher flexural strength, reason behind is steel fibres arrest the crack propagation and changes the path of crack propagation which results into higher load carrying capacity.

Percentage loss in flexural strength for M60(PCC) & M60(FRC) mixes have been observed as 10.37% & 6.09% for 300°C, 26.36% & 17.84% for 500°C, 46.50% & 28.92% for 700°C & 76.23% & 44.62% for 900°C. Results shows that high strength concrete is very weak in flexure during exposed to 900°C temperature. At 900°C temperature M60(PCC) losses almost 75% of flexural strength. M25(FRC) elements displays higher flexural strength, reason behind is steel fibres arrest the crack propagation and changes the path of crack propagation which results into higher load carrying capacity.

Commonly plain concrete is a brittle material in unheated condition as compared to fibre reinforced concrete. At higher temperature exposure, brittleness of plain concrete increases which results in lesser flexural strength. As concrete is weak in flexure, at

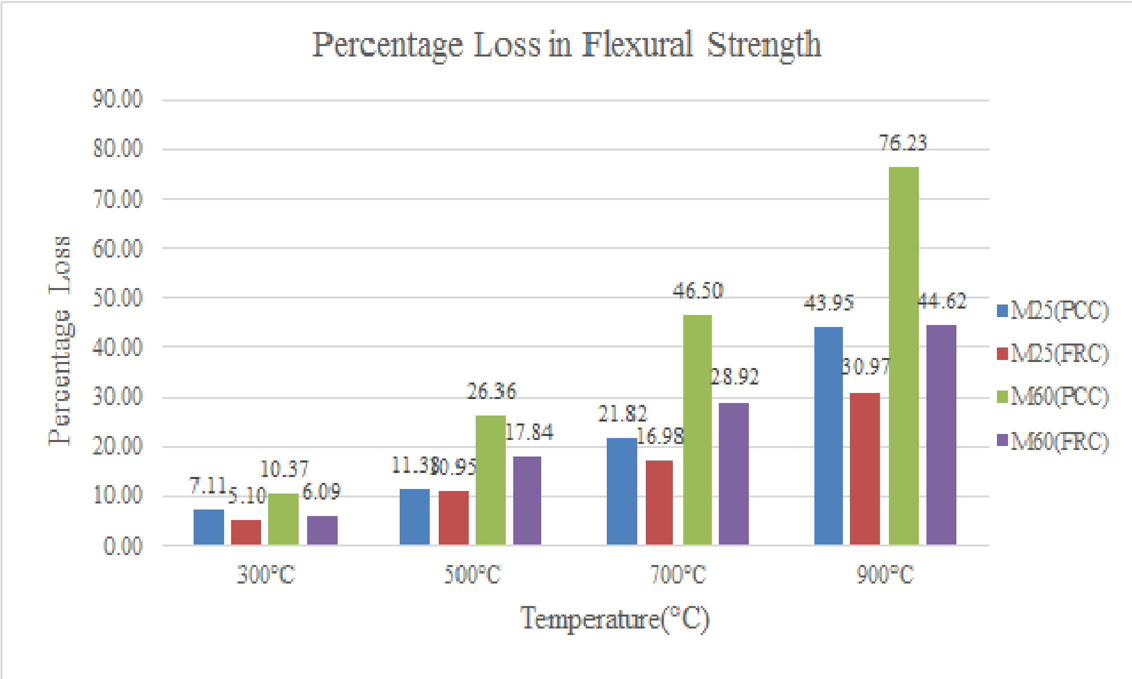


Figure 4.12: % Loss in Flexural Strength at Different Elevated Temperatures as Compared to Unheated Specimens

extreme temperatures PCC elements have very lesser capacity to resist moment which results into lesser flexural strength. Incorporation of steel fibres exhibits ductility to beam specimens which results in higher residual flexural strength. Failure of PCC(M25 & M60) beam elements is of sudden brittle type while for FRC(M25 & M60) beam elements it is gradual ductile.

4.3.4 Modulus of Elasticity

Modulus of Elasticity of all 4 concrete mixes have been tested at the curing age of 28 days and presented in Table 4.6. Modulus of Elasticity of unheated specimens have been kept as base value for comparison purpose. Tangent modulus method has been used to evaluate modulus of elasticity for all concrete mixes. Slope of stress-strain curve gives modulus of elasticity of the concrete. Specimens of all 4 mixes were subjected to different elevated temperature for 60 minutes as per experimental time-temperature curve. Tested values of MoE have been compared with that of unheated specimens.
Sample calculation of modulus of elasticity of M25(PCC) at the age of 28 days has been presented.

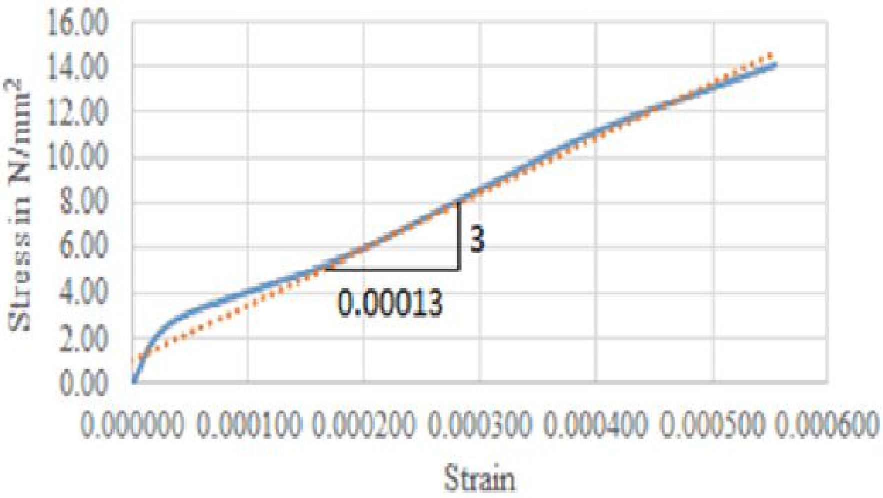


Figure 4.13: Graphical Representation of Stress v/s Strain Relation for M25(PCC) at 28 Days

Modulus of Elasticity $= \frac{Stress}{Strain} = \frac{3}{0.00013} = 23076$ MPa
Percentage loss in MoE for M25(PCC) &M25(FRC) mixes have been observed as 11.85%, 6.69% for 300°C, 18.63%, 11.16% for 500°C, 28.03%, 20.10% for 700°C & 41.29%, 29.20% for 900°C as presented in Figure 4.14

Percentage loss in MoE for M60(PCC) & M60(FRC) mixes have been observed as 12.92% & 6.89% for 300°C, 21.27% & 14.5% for 500°C, 34.88% & 25% for 700°C & 51.96% & 40.58% for 900°C.

Modulus of Elasticity is related with ductility. FRC elements have higher ductility compared to PCC elements for NSC & HSC, hence FRC elements displays higher value of MoE compare to PCC.

Table 4.6: Residual Modulus of Elasticity of All Mixes Exposed to Different Elevated Temperatures

Temp. (°C)	M25(PCC)		M25(FRC)		M60(PCC)		M60(FRC)	
	MOE (MPa)	Avg. (MPa)	MOE (MPa)	Avg. (MPa)	MOE (MPa)	Avg. (MPa)	MOE (MPa)	Avg. (MPa)
Unheated	22364	23223	26445	25472	36887	36154	39568	39334
	24230		24850		36125		38545	
	23076		25120		35450		39890	
300°C	21416	20471	24656	23767	32225	31483	37259	36625
	19874		23895		31467		36153	
	20123		22751		30756		36462	
500°C	20865	18897	22875	22628	27845	28465	32154	33627
	18314		23136		28165		34847	
	17512		21874		29385		33879	
700°C	18678	16714	21246	20353	23557	23544	29879	29499
	15345		20256		22948		28467	
	16120		19557		24126		30152	
900°C	13540	13634	19114	18035	16257	17367	23571	23371
	14785		17135		17489		22387	
	12578		17856		18356		24156	

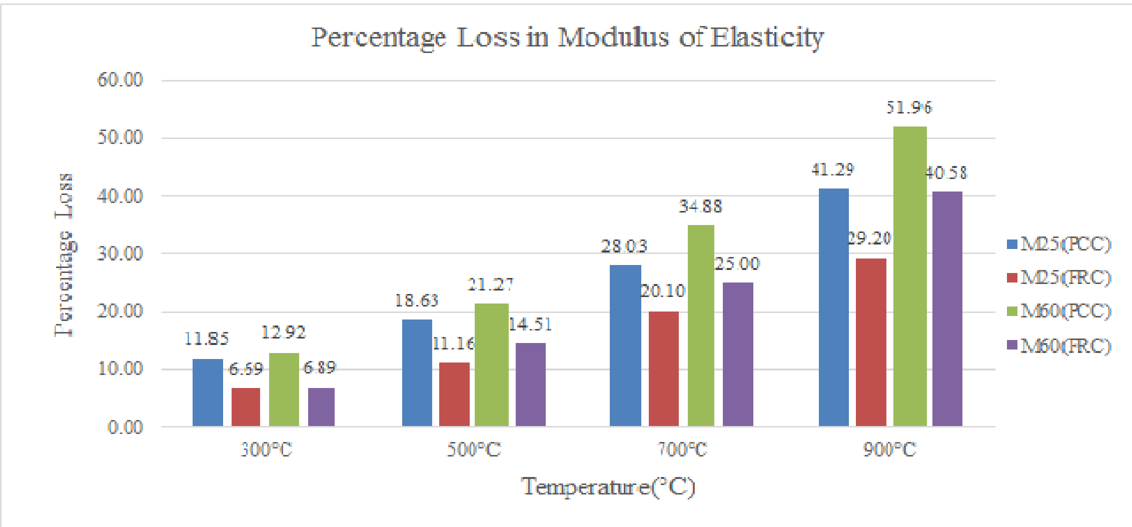


Figure 4.14: % Loss in Modulus of Elasticity at Different Elevated Temperatures as Compared to Unheated Specimens

4.3.5 Bond Strength

The bond strength of all 4 concrete mixes have been tested at the curing age of 28 days and presented in Table 4.7. Bond strength of unheated specimens have been kept as base value for comparison purpose. Average values of 3 specimens have been taken as final result. Bond cube Specimens of all 4 mixes were subjected to different elevated temperature for 60 minutes as per experimental time-temperature curve. Tested values of bond strength have been compared with that of unheated specimens.

Load-slip curve of all four concrete mixes at the age of 28 days is presented in Figure 4.15.

Sample calculation for bond strength evaluation of M25(PCC) exposed different elevated temperature has been given.

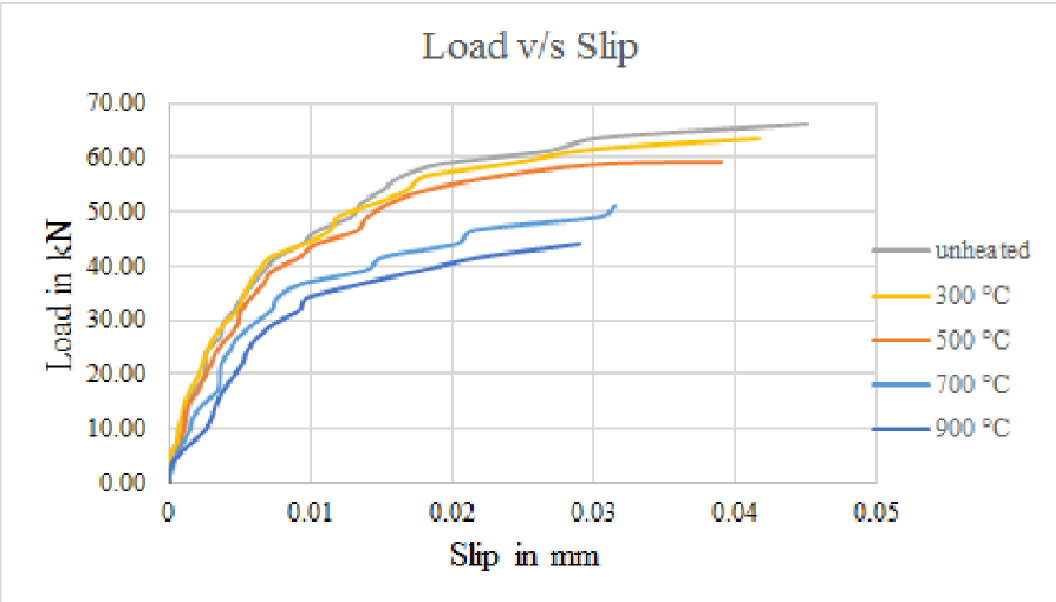


Figure 4.15: Load-Slip Curve for M25(PCC) Subjected to Different Elevated Temperatures

$$\text{Bond Strength} = \frac{FailureLoad}{Embeddedsurfaceareaofbar} = \frac{69170}{3.14 \times 150 \times 12} = 12.23 \text{ MPa}$$

Percentage loss in bond strength for M25(PCC) & M25(FRC) mixes have been observed as 7.57%, 6.08% for 300°C, 12.33%, 8.79% for 500°C, 24.79%, 21.04% for 700°C & 37.48%, 34.51%, for 900°C.

Percentage loss in bond strength for M60(PCC) & M60(FRC) mixes have been observed as 6.96% & 8.38% for 300°C, 15.5% & 17.02% for 500°C, 21.07% & 25.98% for 700°C & 30.52% & 34.50% for 900°C.

Table 4.7: Residual Bond Strength of All Mixes Exposed to Different Elevated Temperatures

Temp. (°C)	M25(PCC)		M25(FRC)		M60(PCC)		M60(FRC)	
	Bond Stren. (MPa)	Avg. (MPa)	Bond Stren. (MPa)	Avg. (MPa)	Bond Stren. (MPa)	Avg. (MPa)	Bond Stren. (MPa)	Avg. (MPa)
Unheated	11.50	11.83	13.10	13.04	14.64	14.18	14.89	14.59
	11.75		12.12		13.58		14.68	
	12.23		13.90		14.31		14.20	
300°C	11.20	10.93	11.56	12.25	14.32	13.19	13.91	13.37
	11.35		12.88		12.80		13.50	
	10.24		12.30		12.45		12.70	
500°C	10.50	10.37	12.20	11.89	12.34	11.97	12.54	12.11
	10.82		11.78		11.68		11.66	
	9.78		11.70		11.90		12.12	
700°C	9.03	8.89	10.87	10.30	11.45	11.19	10.74	10.80
	8.10		10.57		11.25		10.34	
	9.55		9.45		10.87		11.32	
900°C	7.81	7.39	8.12	8.54	10.56	9.85	10.20	9.56
	7.35		8.40		9.87		9.36	
	7.02		9.10		9.12		9.11	

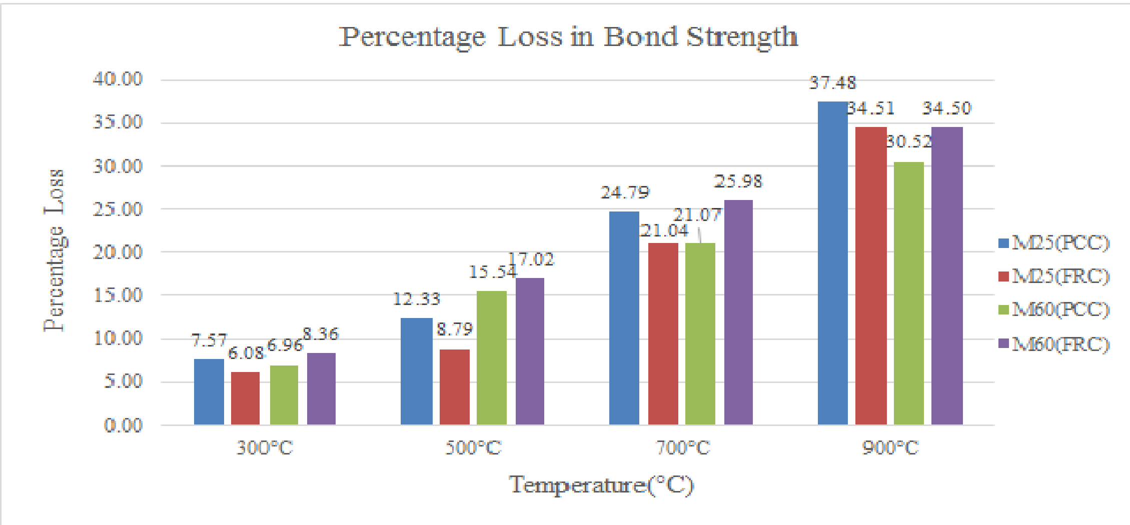


Figure 4.16: % Loss in Bond Strength at Different Elevated Temperatures as Compared to Unheated Specimens

Results shows that there no any significant effect of steel fibres on bond strength at elevated temperatures. Normal strength concrete & High strength concrete performs almost in equal manner. Percentage loss in bond strength for Normal strength concrete & high strength concrete exposed to elevated temperature is more or less equal.

Chapter 5

Results of RC Elements & Discussion

5.1 General

This chapter deals with reporting of test results like: Ultimate failure load, deflection and strain of various types of columns(heated & Unheated). Load on the column was increased at specific intervals and corresponding to every load deflection and lateral strains were measured for the columns. Comparison of Ultimate failure load, maximum deflection, lateral strain and axial strain evaluated at different positions for all categories of columns is presented in tabular as well as in graphical form. These parameters are very essential to understand the behavior of all the columns. Different parameters discussed in this chapter for RC columns are as follows:

- Ultimate failure load
- Load v/s deflection relationship
- Axial stress v/s lateral strain relationship
- Failure modes & crack patterns

5.2 Ultimate Failure Load

Cube compressive strength at the age of 28 days has been measured for each column at the time of testing is presented in Table 5.1.

Table 5.1: Cube Compressive Strength

Specimen	M25(PCC)	M25(FRC)	M60(PCC)	M60(FRC)
1	31.87	33.21	62.56	64.78
2	32.45	32.78	61.32	63.45
3	32.89	34.15	63.1	64.21
Avg. Strength (MPa)	32.40	33.38	62.33	64.15

Interval for load increment was taken as 20 kN for M25 grade specimens and 30 kN for M60 grade specimens. This interval was kept constant up-to complete failure of the column specimen. Experimental average (of 2 columns of same mix) failure load for each type of RC columns are give in Table 5.2

Table 5.2: % Difference in Experimental Ultimate Load of Column to that of Theoretical Value

Column type	Specimen	Experimental Ultimate Failure Load	Avg. Experimental Ultimate Failure Load	Theoretical Ultimate Failure Load	% Difference in Failure load compared to Theoretical Load
		Load (kN)	Load (kN)	Load (kN)	(%)
M25 PCC (Unheated)	1	465	471	337	+39.61
	2	476			
M25 PCC (Heated)	1	352	359	337	+6.53
	2	366			
M25 FRC (Unheated)	1	494	502	339	+48.08
	2	510			
M25 FRC (Heated)	1	393	410	339	+20.94
	2	427			
M60 PCC (Unheated)	1	725	738	645	+14.42
	2	751			
M60 PCC (Heated)	1	594	606	645	-6.04
	2	618			
M60 FRC (Unheated)	1	795	810	649	+24.73
	2	824			
M60 FRC (Heated)	1	660	678	649	+4.46
	2	696			

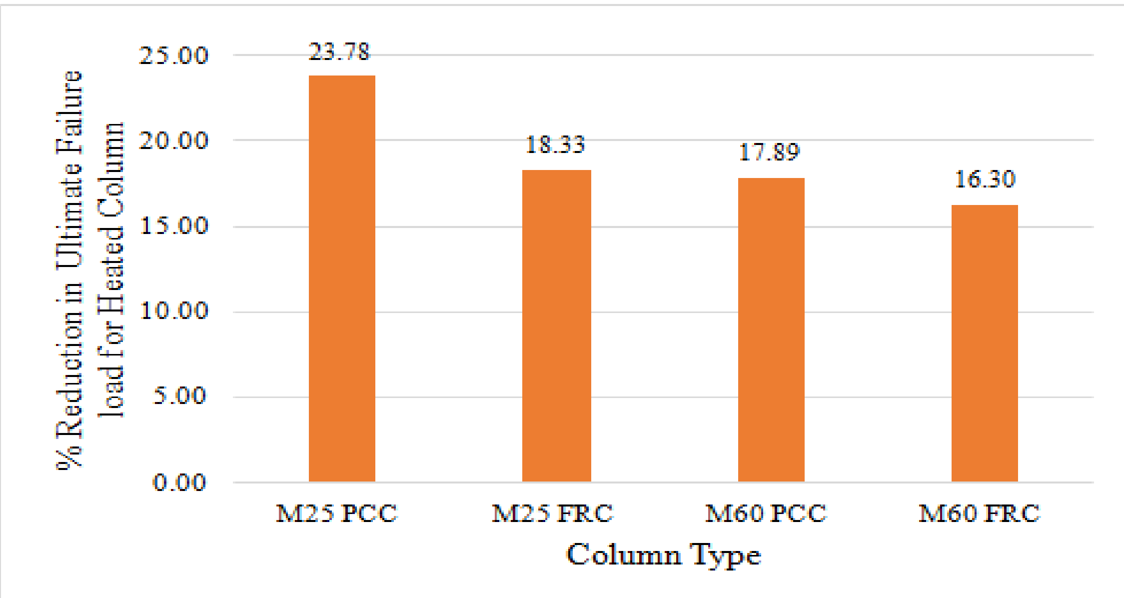


Figure 5.1: Percentage Reduction in Ultimate Failure Load for Heated Columns

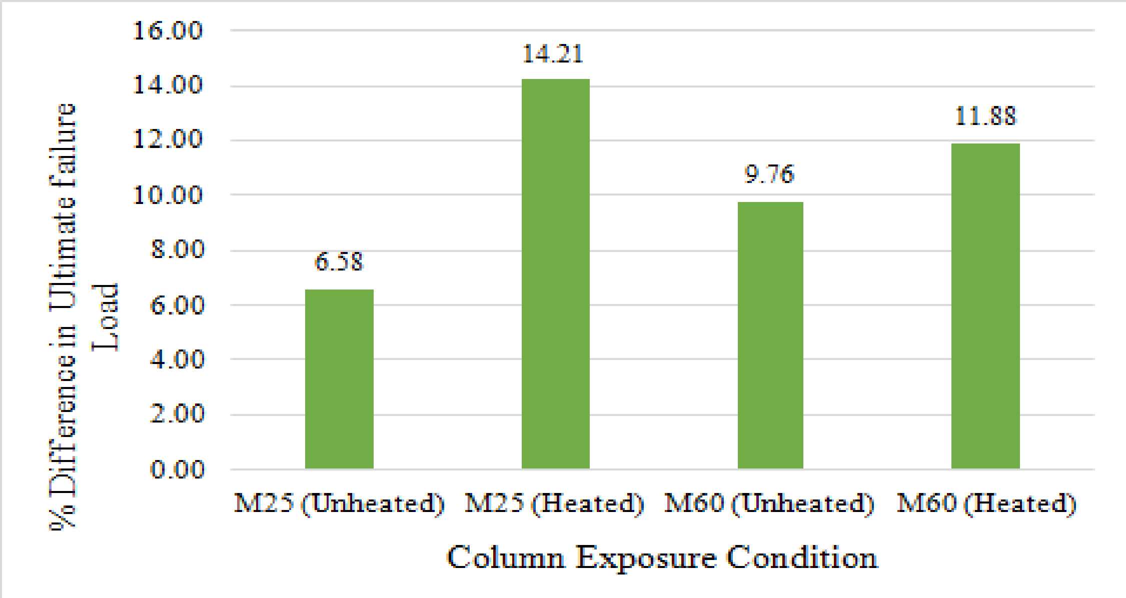


Figure 5.2: Percentage Difference in Ultimate Failure Load of FRC Column to that of Plain Concrete Column

It has been observed from Table 5.2 that percentage difference in experimental failure load compared to theoretical failure value for M25(PCC), M25(FRC), M60(PCC) & M60(FRC) columns in Unheated conditions are +39.61%, +48.08%, +14.42% & +24.73% respectively. For Heated condition these values are +6.53%, +20.94%, -6.04% & +4.46% respectively.

Figure 5.1 presents percentage reduction in ultimate failure load for heated columns to that of unheated columns. These reductions for M25(PCC), M25(FRC), M60(PCC) & M60(FRC) are 23.78%, 18.33%, 17.89% & 16.30% respectively.

Figure 5.2 presents percentage difference in ultimate failure load of FRC column to that of Plain concrete columns. This percentage difference in ultimate load carrying capacity for M25(Unheated), M25(Heated), M60(Unheated) & M60(Heated) are 6.58%, 14.21%, 9.76% & 11.88% respectively. Result shows that FRC columns have higher ultimate load carrying capacity as compared to plain concrete mix columns.

5.3 Load v/s Deflection Relationship

Deflection was measured along the height of the columns. The gauge length of the columns for measuring the deflection was kept 650 mm. To set the LVDT for measuring the deflection of the column, steel frame set-up was prepared. Deflection of M25 specimens were measured at the load interval of 20 kN and for M60 mixes at 30 kN load interval. The results of all column specimens for load and deflection are presented in Table 5.3 to Table 5.10.

Table 5.3: Load & Displacement for M25(PCC) Unheated Column

M25(PCC)Unheated							
Load (kN)	Deflection (mm)		Load (kN)	Deflection (mm)		Load (kN)	Avg. Deflection (mm)
0	0.00		0	0.00		0	0.00
20	0.00		20	0.00		20	0.00
40	0.00		40	0.00		40	0.00
60	0.10		60	0.10		60	0.10
80	0.20		80	0.20		80	0.20
100	0.20		100	0.30		100	0.25
120	0.30		120	0.40		120	0.35
140	0.40		140	0.40		140	0.40
160	0.40		160	0.50		160	0.45
180	0.50		180	0.60		180	0.55
200	0.50		200	0.60		200	0.55
220	0.60		220	0.70		220	0.65
240	0.60		240	0.70		240	0.65
260	0.70		260	0.80		260	0.75
280	0.70		280	0.80		280	0.75
300	0.80		300	0.90		300	0.85
320	0.80		320	0.90		320	0.85
340	0.90		340	1.00		340	0.95
360	0.90		360	1.00		360	0.95
380	1.10		380	1.10		380	1.10
400	1.10		400	1.20		400	1.15
420	1.20		420	1.30		420	1.25
440	1.30		440	1.40		440	1.35
460	1.50		460	1.60		460	1.55
465	-		476	-		470.5	-

Table 5.3 represents load v/s displacement value of M25(PCC) column in Unheated condition. Avg. load & deflection values have been taken for final deflection value. Ultimate load for column 1 & 2 are 465 kN & 476 kN respectively. Deflection values measured at 460 kN are 1.50 mm & 1.60 mm respectively. Avg. ultimate load is 470.5 kN.

Table 5.4: Load & Displacement for M25(PCC) Heated Column

M25(PCC)Heated							
Load (kN)	Deflection (mm)		Load (kN)	Deflection (mm)		Load (kN)	Avg. Deflection (mm)
0	0.00		0	0.00		0	0.00
20	0.10		20	0.10		20	0.10
40	0.10		40	0.20		40	0.15
60	0.20		60	0.30		60	0.25
80	0.30		80	0.40		80	0.35
100	0.40		100	0.40		100	0.40
120	0.50		120	0.50		120	0.50
140	0.50		140	0.60		140	0.55
160	0.60		160	0.70		160	0.65
180	0.60		180	0.80		180	0.70
200	0.70		200	0.90		200	0.80
220	0.70		220	1.00		220	0.85
240	0.70		240	1.00		240	0.85
260	0.80		260	1.10		260	0.95
280	0.90		280	1.10		280	1.00
300	1.00		300	1.20		300	1.10
320	1.10		320	1.20		320	1.15
340	1.30		340	1.40		340	1.35
352	-		366	-		359	-

Table 5.4 represents load v/s displacement value of M25(PCC) column in heated condition. Avg. load & deflection values have been taken for final deflection value. Ultimate load for column 1 & 2 are 352 kN & 366 kN respectively. Deflection values measured at 340 kN are 1.30 mm & 1.40 mm respectively. Avg. ultimate load is 359 kN.

Table 5.5: Load & Displacement for M25(FRC) Unheated Column

M25(FRC)Unheated							
Load (kN)	Deflection (mm)		Load (kN)	Deflection (mm)		Load (kN)	Avg. Deflection (mm)
0	0.00		0	0.00		0	0.00
20	0.00		20	0.00		20	0.00
40	0.00		40	0.00		40	0.00
60	0.10		60	0.10		60	0.10
80	0.10		80	0.20		80	0.15
100	0.20		100	0.30		100	0.25
120	0.30		120	0.40		120	0.35
140	0.40		140	0.40		140	0.40
160	0.50		160	0.50		160	0.50
180	0.50		180	0.50		180	0.50
200	0.60		200	0.70		200	0.65
220	0.80		220	0.80		220	0.80
240	0.90		240	1.00		240	0.95
260	1.10		260	1.10		260	1.10
280	1.20		280	1.30		280	1.25
300	1.40		300	1.50		300	1.45
320	1.60		320	1.70		320	1.65
340	1.80		340	1.90		340	1.85
360	2.00		360	2.10		360	2.05
380	2.20		380	2.30		380	2.25
400	2.50		400	2.50		400	2.50
420	2.80		420	2.90		420	2.85
440	3.20		440	3.20		440	3.20
460	3.50		460	3.60		460	3.55
480	4.00		480	3.90		480	3.95
494	-		510	-		502	-

Table 5.5 represents load v/s displacement value of M25(FRC) column in Unheated condition. Avg. load & deflection values have been taken for final deflection value. Ultimate load for column 1 & 2 are 494 kN & 510 kN respectively. Deflection values measured at 480 kN are 4 mm & 3.90 mm respectively. Avg. ultimate load is 502 kN.

Table 5.6: Load & Displacement for M25(FRC) Heated Column

M25(FRC)Heated							
Load (kN)	Deflection (mm)		Load (kN)	Deflection (mm)		Load (kN)	Deflection (mm)
0	0.00		0	0.00		0	0.00
20	0.00		20	0.00		20	0.00
40	0.10		40	0.10		40	0.10
60	0.20		60	0.30		60	0.25
80	0.30		80	0.40		80	0.35
100	0.40		100	0.50		100	0.45
120	0.60		120	0.60		120	0.60
140	0.70		140	0.80		140	0.75
160	0.80		160	1.00		160	0.90
180	0.90		180	1.20		180	1.05
200	1.00		200	1.40		200	1.20
220	1.30		220	1.60		220	1.45
240	1.60		240	1.70		240	1.65
260	1.80		260	1.90		260	1.85
280	2.10		280	2.20		280	2.15
300	2.50		300	2.40		300	2.45
320	2.70		320	2.70		320	2.70
340	2.90		340	3.00		340	2.95
360	3.30		360	3.20		360	3.25
380	3.40		380	3.50		380	3.45
393	-		427	-		410	-

Table 5.6 represents load v/s displacement value of M25(FRCC) column in heated condition. Avg. load & deflection values have been taken for final deflection value. Ultimate load for column 1 & 2 are 393 kN & 427 kN respectively. Deflection values measured at 380 kN are 3.40 mm & 3.50 mm respectively. Avg. ultimate load is 410 kN.

Table 5.7: Load & Displacement for M60(PCC) Unheated Column

M60(PCC)Unheated							
Load (kN)	Deflection (mm)		Load (kN)	Deflection (mm)		Load (kN)	Avg. Deflection (mm)
0	0.00		0	0.00		0	0.00
30	0.00		30	0.00		30	0.00
60	0.10		60	0.00		60	0.05
90	0.20		90	0.10		90	0.15
120	0.30		120	0.20		120	0.25
150	0.40		150	0.30		150	0.35
180	0.50		180	0.40		180	0.45
210	0.50		210	0.40		210	0.45
240	0.60		240	0.50		240	0.55
270	0.70		270	0.60		270	0.65
300	0.80		300	0.60		300	0.70
330	0.70		330	0.70		330	0.70
360	0.80		360	0.80		360	0.80
390	0.80		390	0.90		390	0.85
420	0.90		420	0.90		420	0.90
450	1.10		450	1.10		450	1.10
480	1.10		480	1.20		480	1.15
510	1.30		510	1.30		510	1.30
540	1.50		540	1.50		540	1.50
570	1.70		570	1.70		570	1.70
600	1.90		600	1.90		600	1.90
630	2.10		630	2.10		630	2.10
660	2.30		660	2.30		660	2.30
690	2.50		690	2.50		690	2.50
720	2.70		720	2.70		720	2.70
725	-		751	-		738	-

Table 5.7 represents load v/s displacement value of M60(PCC) column in Unheated condition. Avg. load & deflection values have been taken for final deflection value. Ultimate load for column 1 & 2 are 725 kN & 751 kN respectively. Deflection values measured at 720 kN are 2.70 mm & 2.70 mm respectively. Avg. ultimate load is 738 kN.

Table 5.8: Load & Displacement for M60(PCC) Heated Column

M60(PCC)Heated							
Load (kN)	Deflection (mm)		Load (kN)	Deflection (mm)		Load (kN)	Deflection (mm)
0	0.00		0	0.00		0	0.00
30	0.10		30	0.10		30	0.10
60	0.10		60	0.20		60	0.15
90	0.20		90	0.30		90	0.25
120	0.30		120	0.40		120	0.35
150	0.50		150	0.40		150	0.45
180	0.60		180	0.50		180	0.55
210	0.80		210	0.60		210	0.70
240	0.90		240	0.70		240	0.80
270	1.00		270	0.80		270	0.90
300	1.10		300	0.90		300	1.00
330	1.20		330	1.00		330	1.10
360	1.30		360	1.10		360	1.20
390	1.40		390	1.10		390	1.25
420	1.50		420	1.20		420	1.35
450	1.60		450	1.40		450	1.50
480	1.70		480	1.40		480	1.55
510	1.80		510	1.60		510	1.70
540	2.00		540	1.80		540	1.90
570	2.10		570	1.90		570	2.00
594	-		618	-		606	-

Table 5.8 represents load v/s displacement value of M60(PCC) column in heated condition. Avg. load & deflection values have been taken for final deflection value. Ultimate load for column 1 & 2 are 594 kN & 618 kN respectively. Deflection values measured at 570 kN are 2.10 mm & 1.90 mm respectively. Avg. ultimate load is 606 kN.

Table 5.9: Load & Displacement for M60(FRC) Unheated Column

M60(FRC)Unheated							
Load (kN)	Deflection (mm)		Load (kN)	Deflection (mm)		Load (kN)	Avg. Deflection (mm)
0	0.00		0	0.00		0	0.00
30	0.00		30	0.00		30	0.00
60	0.00		60	0.00		60	0.00
90	0.10		90	0.10		90	0.10
120	0.10		120	0.20		120	0.15
150	0.20		150	0.30		150	0.25
180	0.30		180	0.40		180	0.35
210	0.40		210	0.40		210	0.40
240	0.50		240	0.50		240	0.50
270	0.50		270	0.50		270	0.50
300	0.60		300	0.60		300	0.60
330	0.60		330	0.60		330	0.60
360	0.70		360	0.70		360	0.70
390	0.70		390	0.70		390	0.70
420	0.80		420	0.90		420	0.85
450	0.90		450	1.00		450	0.95
480	1.00		480	1.10		480	1.05
510	1.00		510	1.20		510	1.10
540	1.10		540	1.30		540	1.20
570	1.20		570	1.40		570	1.30
600	1.40		600	1.60		600	1.50
630	1.60		630	1.70		630	1.65
660	2.00		660	1.90		660	1.95
690	2.40		690	2.30		690	2.35
720	3.00		720	2.80		720	2.90
750	3.40		750	3.30		750	3.35
780	3.90		780	3.60		780	3.75
795	-		824	-		810	-

Table 5.9 represents load v/s displacement value of M60(FRC) column in Unheated condition. Avg. load & deflection values have been taken for final deflection value. Ultimate load for column 1 & 2 are 795 kN & 824 kN respectively. Deflection values measured at 780 kN are 3.90 mm & 3.60 mm respectively. Avg. ultimate load is 810 kN.

Table 5.10: Load & Displacement for M60(FRC) Heated Column

M60(FRC)Heated							
Load (kN)	Deflection (mm)		Load (kN)	Deflection (mm)		Load (kN)	Deflection (mm)
0	0.00		0	0.00		0	0.00
30	0.00		30	0.00		30	0.00
60	0.10		60	0.10		60	0.10
90	0.20		90	0.30		90	0.25
120	0.20		120	0.40		120	0.30
150	0.40		150	0.50		150	0.45
180	0.60		180	0.60		180	0.60
210	0.70		210	0.70		210	0.70
240	0.80		240	0.80		240	0.80
270	0.90		270	0.80		270	0.85
300	1.00		300	0.70		300	0.85
330	1.10		330	0.70		330	0.90
360	1.20		360	0.80		360	1.00
390	1.20		390	0.80		390	1.00
420	1.30		420	0.90		420	1.10
450	1.30		450	1.20		450	1.25
480	1.40		480	1.30		480	1.35
510	1.50		510	1.40		510	1.45
540	1.60		540	1.80		540	1.70
570	1.80		570	2.20		570	2.00
600	2.40		600	2.60		600	2.50
630	2.90		630	3.10		630	3.00
660	3.50		660	3.50		660	3.50
660	-		696	-		678	-

Table 5.10 represents load v/s displacement value of M60(FRC) column in heated condition. Avg. load & deflection values have been taken for final deflection value. Ultimate load for column 1 & 2 are 660 kN & 696 kN respectively. Deflection values measured at 460 kN are 3.50 mm & 3.50 mm respectively. Avg. ultimate load is 470.5 kN.

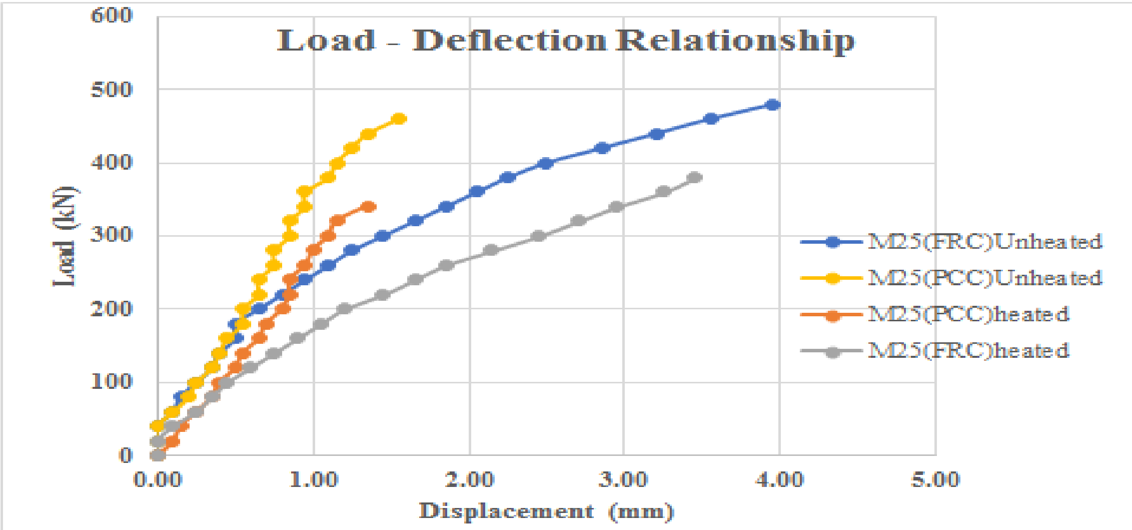


Figure 5.3: Load v/s Deflection Relationship(M25)

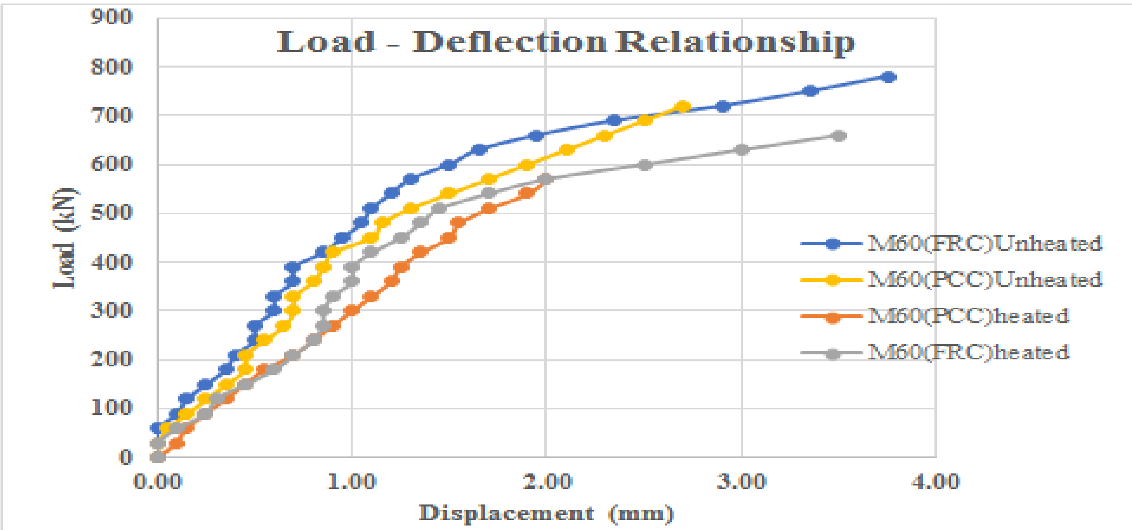


Figure 5.4: Load v/s Deflection Relationship(M60)

Figure 5.3 & 5.4 represents graphical representation of load v/s displacement for M25 grade & M60 grade concrete mixes respectively. For Normal strength concrete Column produced from M25(PCC) grade columns have less deflection and hence less load carrying capacity as compared to M25(FRC) & M60(FRC) grade respectively. At elevated temperature M25(PCC) & M60(PCC) becomes more brittle which results into sudden brittle failure while FRC mixes are more ductile as compare to PCC mix.

Table 5.11 presents displacement ductility values for all mixes having two exposure conditions, heated & unheated. Displacement ductility has been found out by maximum displacement divided by yield displacement. Result shows that displacement ductility for columns having plain concrete have lesser value as compared to fibre reinforced column. Result shows that after heating the columns, ductility reduces as compare to that of unheated specimens.

Table 5.11: Displacement Ductility

Column Type	Avg. Yield Load (kN)	Avg.Yield Displ. (mm)	Avg.Ultimate Load (kN)	Avg. Max. Displ. (mm)	Displacement Ductility
M25 (PCC) Unheated	370	1	471	1.55	1.55
M25 (PCC) Heated	250	0.9	359	1.35	1.50
M25 (FRC) Unheated	300	1.45	502	3.95	2.72
M25 (FRC) Heated	200	1.2	410	3.45	2.88
M60 (PCC) Unheated	600	1.5	738	2.7	1.80
M60 (PCC) Heated	390	1.25	606	2	1.60
M60 (FRC) Unheated	570	1.3	810	3.75	2.88
M60 (FRC) Heated	510	1.45	678	3.5	2.41

5.4 Axial Stress v/s Strain Relationship

Compressive stress is axial stress that tends to cause a body to become shorter along the direction of applied force. It is the load divided by the cross sectional area and lateral strain was measured by mechanical strain gauges at the mid-height. Results for axial stress and lateral strain of all columns are presented in Table 5.12 to Table 5.19.

Table 5.12: Axial Stress and Lateral Strain for Reinforced M25(PCC) Unheated Column

M25(PCC)Unheated							
Stress (MPa)	Strain		Stress (MPa)	Strain		Stress (MPa)	Avg. Strain
0.00	0.0000		0.00	0.0000		0.00	0.0000
0.89	0.0000		0.89	0.0000		0.89	0.0000
1.78	0.0000		1.78	0.0000		1.78	0.0000
2.67	0.0002		2.67	0.0000		2.67	0.0001
3.56	0.0003		3.56	0.0001		3.56	0.0002
4.44	0.0004		4.44	0.0001		4.44	0.0003
5.33	0.0004		5.33	0.0001		5.33	0.0003
6.22	0.0005		6.22	0.0002		6.22	0.0004
7.11	0.0006		7.11	0.0002		7.11	0.0004
8.00	0.0006		8.00	0.0003		8.00	0.0005
8.89	0.0006		8.89	0.0004		8.89	0.0005
9.78	0.0007		9.78	0.0005		9.78	0.0006
10.67	0.0008		10.67	0.0006		10.67	0.0007
11.56	0.0009		11.56	0.0008		11.56	0.0009
12.44	0.0009		12.44	0.0009		12.44	0.0009
13.33	0.0010		13.33	0.0010		13.33	0.0010
14.22	0.0011		14.22	0.0012		14.22	0.0012
15.11	0.0012		15.11	0.0014		15.11	0.0013
16.00	0.0015		16.00	0.0016		16.00	0.0016
16.89	0.0017		16.89	0.0018		16.89	0.0018
17.78	0.0019		17.78	0.0020		17.78	0.0020
18.67	0.0021		18.67	0.0022		18.67	0.0022
19.56	0.0023		19.56	0.0023		19.56	0.0023
20.44	0.0025		20.44	0.0024		20.44	0.0025

Table 5.12 represents axial stress v/s lateral strain values for M25(PCC) unheated column. At axial stress value of 20.44 MPa, lateral strain for column 1 & 2 is 0.0025 & 0.0024 respectively.

Table 5.13: Axial Stress and Lateral Strain for Reinforced M25(PCC) Heated Column

M25(PCC)Heated							
Stress (MPa)	Strain		Stress (MPa)	Strain		Avg. Stress (MPa)	Avg. Strain
0.00	0.0000		0.00	0.0000		0.00	0.0000
0.89	0.0001		0.89	0.0001		0.89	0.0001
1.78	0.0001		1.78	0.0002		1.78	0.0002
2.67	0.0002		2.67	0.0003		2.67	0.0003
3.56	0.0002		3.56	0.0004		3.56	0.0003
4.44	0.0003		4.44	0.0004		4.44	0.0004
5.33	0.0004		5.33	0.0005		5.33	0.0005
6.22	0.0005		6.22	0.0006		6.22	0.0006
7.11	0.0006		7.11	0.0007		7.11	0.0007
8.00	0.0007		8.00	0.0008		8.00	0.0008
8.89	0.0008		8.89	0.0009		8.89	0.0009
9.78	0.0009		9.78	0.0010		9.78	0.0010
10.67	0.0010		10.67	0.0012		10.67	0.0011
11.56	0.0011		11.56	0.0013		11.56	0.0012
12.44	0.0012		12.44	0.0014		12.44	0.0013
13.33	0.0014		13.33	0.0016		13.33	0.0015
14.22	0.0017		14.22	0.0018		14.22	0.0018
15.11	0.0019		15.11	0.0020		15.11	0.0020

Table 5.13 represents axial stress v/s lateral strain values for M25(PCC) heated column. At axial stress value of 15.11 MPa, lateral strain for column 1 & 2 is 0.0019 & 0.0020 respectively.

Table 5.14: Axial Stress and Lateral Strain for Reinforced M25(FRC) Unheated Column

M25(FRC)Unheated							
Stress (MPa)	Strain		Stress (MPa)	Strain		Avg. Stress (MPa)	Avg. Strain
0.00	0.0000		0.00	0.0000		0.00	0.0000
0.89	0.0000		0.89	0.0000		0.89	0.0000
1.78	0.0000		1.78	0.0000		1.78	0.0000
2.67	0.0001		2.67	0.0000		2.67	0.0001
3.56	0.0002		3.56	0.0001		3.56	0.0002
4.44	0.0002		4.44	0.0003		4.44	0.0003
5.33	0.0003		5.33	0.0004		5.33	0.0004
6.22	0.0003		6.22	0.0004		6.22	0.0004
7.11	0.0004		7.11	0.0005		7.11	0.0005
8.00	0.0004		8.00	0.0005		8.00	0.0005
8.89	0.0004		8.89	0.0005		8.89	0.0005
9.78	0.0005		9.78	0.0006		9.78	0.0006
10.67	0.0005		10.67	0.0006		10.67	0.0006
11.56	0.0005		11.56	0.0007		11.56	0.0006
12.44	0.0007		12.44	0.0007		12.44	0.0007
13.33	0.0008		13.33	0.0007		13.33	0.0008
14.22	0.0009		14.22	0.0008		14.22	0.0009
15.11	0.0015		15.11	0.0013		15.11	0.0014
16.00	0.0017		16.00	0.0016		16.00	0.0017
16.89	0.0020		16.89	0.0019		16.89	0.0020
17.78	0.0024		17.78	0.0022		17.78	0.0023
18.67	0.0028		18.67	0.0031		18.67	0.0030
19.56	0.0032		19.56	0.0035		19.56	0.0034
20.44	0.0036		20.44	0.0038		20.44	0.0037
21.33	0.0040		21.33	0.0042		21.33	0.0041

Table 5.14 represents axial stress v/s lateral strain values for M25(FRC) unheated column. At axial stress value of 21.33 MPa, lateral strain for column 1 & 2 is 0.0040 & 0.0042 respectively.

Table 5.15: Axial Stress and Lateral Strain for Reinforced M25(FRC) Heated Column

M25(FRC)Heated							
Stress (MPa)	Strain		Stress (MPa)	Strain		Stress (MPa)	Avg. Strain
0.00	0.0000		0.00	0.0000		0.00	0.0000
0.89	0.0001		0.89	0.0001		0.89	0.0001
1.78	0.0001		1.78	0.0001		1.78	0.0001
2.67	0.0002		2.67	0.0002		2.67	0.0002
3.56	0.0002		3.56	0.0002		3.56	0.0002
4.44	0.0003		4.44	0.0002		4.44	0.0003
5.33	0.0004		5.33	0.0003		5.33	0.0004
6.22	0.0005		6.22	0.0004		6.22	0.0005
7.11	0.0006		7.11	0.0004		7.11	0.0005
8.00	0.0006		8.00	0.0005		8.00	0.0006
8.89	0.0008		8.89	0.0006		8.89	0.0007
9.78	0.0010		9.78	0.0007		9.78	0.0009
10.67	0.0013		10.67	0.0009		10.67	0.0011
11.56	0.0016		11.56	0.0011		11.56	0.0014
12.44	0.0019		12.44	0.0015		12.44	0.0017
13.33	0.0021		13.33	0.0021		13.33	0.0021
14.22	0.0023		14.22	0.0024		14.22	0.0024
15.11	0.0026		15.11	0.0026		15.11	0.0026
16.00	0.0028		16.00	0.0029		16.00	0.0029
16.89	0.0030		16.89	0.0032		16.89	0.0031

Table 5.15 represents axial stress v/s lateral strain values for M25(FRC) heated column. At axial stress value of 16.89 MPa, lateral strain for column 1 & 2 is 0.0030 & 0.0032 respectively.

Table 5.16: Axial Stress and Lateral Strain for Reinforced M60(PCC) Unheated Column

M60(PCC)Unheated							
Stress (MPa)	Strain		Stress (MPa)	Strain		Avg. Stress (MPa)	Avg. Strain
0.00	0.0000		0.00	0.0000		0.00	0.0000
1.33	0.0000		1.33	0.0000		1.33	0.0000
2.67	0.0000		2.67	0.0000		2.67	0.0000
4.00	0.0002		4.00	0.0000		4.00	0.0001
5.33	0.0003		5.33	0.0001		5.33	0.0002
6.67	0.0004		6.67	0.0001		6.67	0.0003
8.00	0.0004		8.00	0.0001		8.00	0.0003
9.33	0.0005		9.33	0.0002		9.33	0.0004
10.67	0.0006		10.67	0.0002		10.67	0.0004
12.00	0.0006		12.00	0.0003		12.00	0.0005
13.33	0.0006		13.33	0.0004		13.33	0.0005
14.67	0.0007		14.67	0.0005		14.67	0.0006
16.00	0.0008		16.00	0.0005		16.00	0.0007
17.33	0.0009		17.33	0.0005		17.33	0.0007
18.67	0.0009		18.67	0.0006		18.67	0.0008
20.00	0.0010		20.00	0.0010		20.00	0.0010
21.33	0.0011		21.33	0.0012		21.33	0.0012
22.67	0.0012		22.67	0.0014		22.67	0.0013
24.00	0.0015		24.00	0.0016		24.00	0.0016
25.33	0.0017		25.33	0.0018		25.33	0.0018
26.67	0.0019		26.67	0.0020		26.67	0.0020
28.00	0.0021		28.00	0.0022		28.00	0.0022
29.33	0.0023		29.33	0.0023		29.33	0.0023
30.67	0.0025		30.67	0.0024		30.67	0.0025
32.00	0.0027		32.00	0.0026		32.00	0.0027

Table 5.16 represents axial stress v/s lateral strain values for M60(PCC) unheated column. At axial stress value of 32 MPa, lateral strain for column 1 & 2 is 0.0027 & 0.0026 respectively.

Table 5.17: Axial Stress and Lateral Strain for Reinforced M60(PCC) Heated Column

M60(PCC)Heated							
Stress (MPa)	Strain		Stress (MPa)	Strain		Avg. Stress (MPa)	Avg. Strain
0.00	0.0000		0.00	0.0000		0.00	0.0000
1.33	0.0001		1.33	0.0001		1.33	0.0001
2.67	0.0001		2.67	0.0002		2.67	0.0002
4.00	0.0002		4.00	0.0003		4.00	0.0003
5.33	0.0002		5.33	0.0004		5.33	0.0003
6.67	0.0003		6.67	0.0004		6.67	0.0004
8.00	0.0004		8.00	0.0005		8.00	0.0005
9.33	0.0005		9.33	0.0006		9.33	0.0006
10.67	0.0006		10.67	0.0007		10.67	0.0007
12.00	0.0007		12.00	0.0008		12.00	0.0008
13.33	0.0008		13.33	0.0009		13.33	0.0009
14.67	0.0009		14.67	0.0010		14.67	0.0010
16.00	0.0010		16.00	0.0012		16.00	0.0011
17.33	0.0011		17.33	0.0013		17.33	0.0012
18.67	0.0012		18.67	0.0014		18.67	0.0013
20.00	0.0013		20.00	0.0015		20.00	0.0014
21.33	0.0014		21.33	0.0016		21.33	0.0015
22.67	0.0015		22.67	0.0017		22.67	0.0016
24.00	0.0019		24.00	0.0020		24.00	0.0020
25.33	0.0021		25.33	0.0022		25.33	0.0022

Table 5.17 represents axial stress v/s lateral strain values for M60(PCC) heated column. At axial stress value of 25.33 MPa, lateral strain for column 1 & 2 is 0.0021 & 0.0022 respectively.

Table 5.18: Axial Stress and Lateral Strain for Reinforced M60(FRC) Unheated Column

M60(FRC)Unheated							
Stress (MPa)	Strain		Stress (MPa)	Strain		Avg. Stress (MPa)	Avg. Strain
0.00	0.0000		0.00	0.0000		0.00	0.0000
1.33	0.0000		1.33	0.0000		1.33	0.0000
2.67	0.0000		2.67	0.0000		2.67	0.0000
4.00	0.0001		4.00	0.0000		4.00	0.0001
5.33	0.0002		5.33	0.0001		5.33	0.0002
6.67	0.0002		6.67	0.0003		6.67	0.0003
8.00	0.0003		8.00	0.0004		8.00	0.0004
9.33	0.0003		9.33	0.0004		9.33	0.0004
10.67	0.0004		10.67	0.0005		10.67	0.0005
12.00	0.0004		12.00	0.0005		12.00	0.0005
13.33	0.0004		13.33	0.0005		13.33	0.0005
14.67	0.0005		14.67	0.0006		14.67	0.0006
16.00	0.0005		16.00	0.0006		16.00	0.0006
17.33	0.0005		17.33	0.0007		17.33	0.0006
18.67	0.0007		18.67	0.0007		18.67	0.0007
20.00	0.0008		20.00	0.0007		20.00	0.0008
21.33	0.0009		21.33	0.0008		21.33	0.0009
22.67	0.0015		22.67	0.0013		22.67	0.0014
24.00	0.0017		24.00	0.0016		24.00	0.0017
25.33	0.0020		25.33	0.0019		25.33	0.0020
26.67	0.0024		26.67	0.0022		26.67	0.0023
28.00	0.0025		28.00	0.0024		28.00	0.0025
29.33	0.0027		29.33	0.0026		29.33	0.0027
30.67	0.0030		30.67	0.0028		30.67	0.0029
32.00	0.0033		32.00	0.0032		32.00	0.0033
33.33	0.0037		33.33	0.0035		33.33	0.0036
34.67	0.0044		34.67	0.0042		34.67	0.0043

Table 5.18 represents axial stress v/s lateral strain values for M60(FRC) unheated column. At axial stress value of 34.67 MPa, lateral strain for column 1 & 2 is 0.0044 & 0.0042 respectively.

Table 5.19: Axial Stress and Lateral Strain for Reinforced M60(FRC) Heated Column

M60(FRC)Heated							
Stress (MPa)	Strain		Stress (MPa)	Strain		Stress (MPa)	Avg. Strain
0.00	0.0000		0.00	0.0000		0.00	0.0000
1.33	0.0001		1.33	0.0001		1.33	0.0001
2.67	0.0001		2.67	0.0001		2.67	0.0001
4.00	0.0002		4.00	0.0002		4.00	0.0002
5.33	0.0002		5.33	0.0002		5.33	0.0002
6.67	0.0003		6.67	0.0002		6.67	0.0003
8.00	0.0004		8.00	0.0003		8.00	0.0004
9.33	0.0005		9.33	0.0004		9.33	0.0005
10.67	0.0006		10.67	0.0004		10.67	0.0005
12.00	0.0006		12.00	0.0005		12.00	0.0006
13.33	0.0006		13.33	0.0006		13.33	0.0006
14.67	0.0006		14.67	0.0007		14.67	0.0007
16.00	0.0008		16.00	0.0007		16.00	0.0008
17.33	0.0010		17.33	0.0009		17.33	0.0010
18.67	0.0012		18.67	0.0011		18.67	0.0012
20.00	0.0013		20.00	0.0014		20.00	0.0014
21.33	0.0013		21.33	0.0016		21.33	0.0015
22.67	0.0015		22.67	0.0018		22.67	0.0017
24.00	0.0019		24.00	0.0021		24.00	0.0020
25.33	0.0022		25.33	0.0024		25.33	0.0023
26.67	0.0026		26.67	0.0028		26.67	0.0027
28.00	0.0029		28.00	0.0032		28.00	0.0031
29.33	0.0034		29.33	0.0036		29.33	0.0035

Table 5.19 represents axial stress v/s lateral strain values for M60(FRC) heated column. At axial stress value of 29.33 MPa, lateral strain for column 1 & 2 is 0.0034 & 0.0036 respectively.

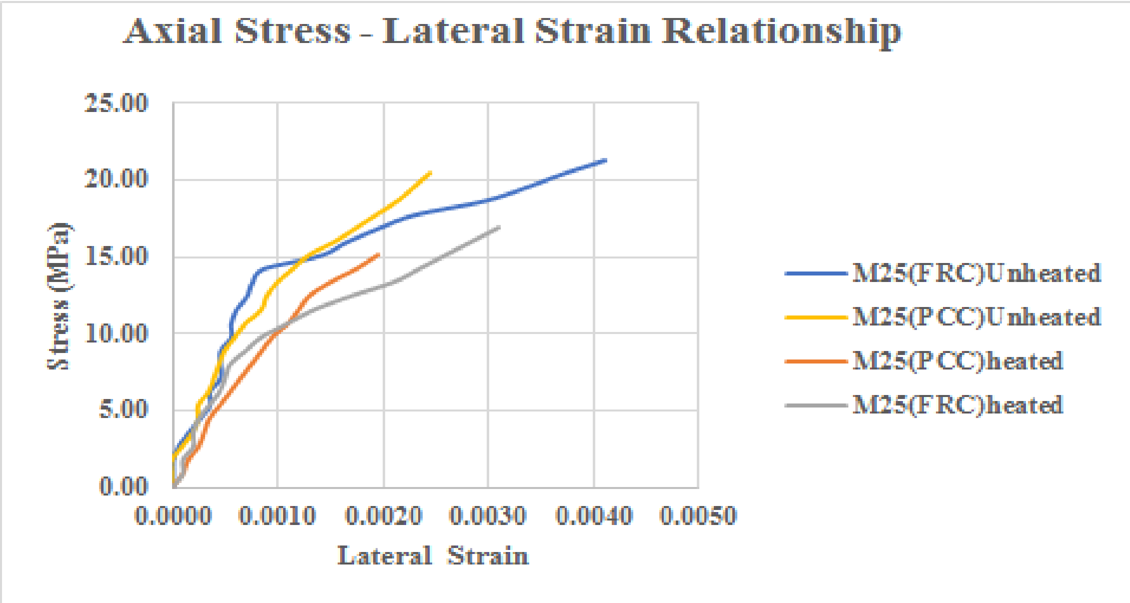


Figure 5.5: Axial Stress v/s Lateral Strain Relationship(M25)

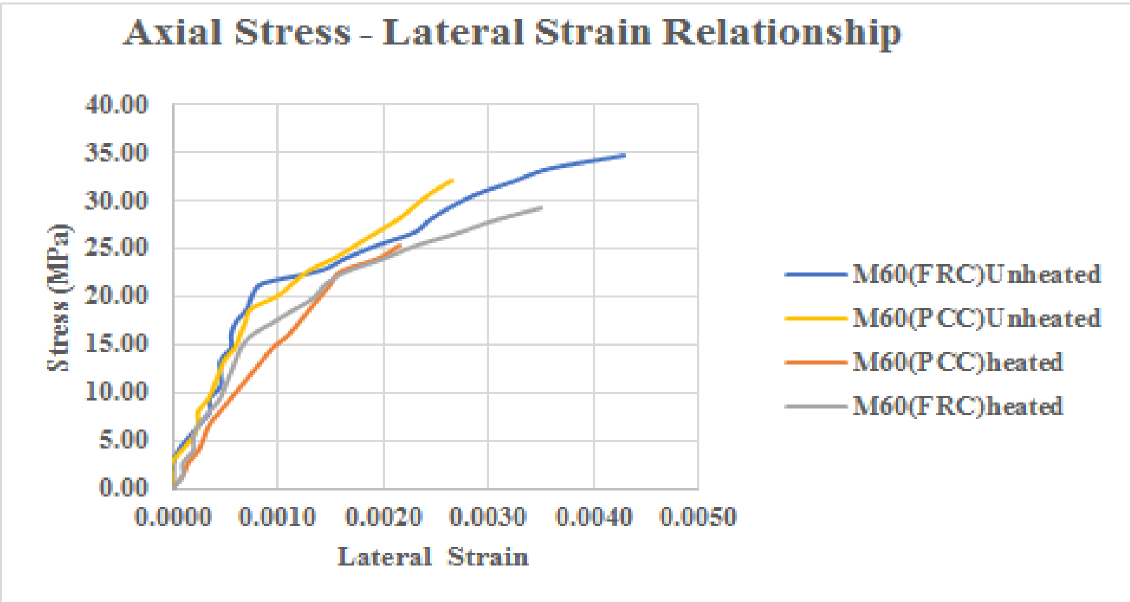


Figure 5.6: Axial Stress v/s Lateral Strain Relationship(M60)

Figure 5.5 & 5.6 represents graphical representation of axial stress v/s lateral strain for M25 & M60 grades respectively. Results shows that M25(PCC) & M60(PCC) are less ductile as compare to M25(FRC) & M60(FRC) respectively for unheated as well as heated conditions.

5.5 Failure Modes & Crack Patterns

RC columns were tested under axial load. RC columns failed when the ultimate compressive strength was gradually increased. Failure mode of all RC column specimens are discussed below:

Reinforced Column(M25-PCC)

Column with M25(PCC) in unheated condition failed from top. Failure was due large amount of stress concentration at top. In Figure 5.7 local concrete crushing as well as buckling of reinforcement can be observed at top in compression. It has been observed that core was intact, but the cover has been damaged upto one third height of the column. As presented in Figure 5.7.

Heated column failed due to heavy concentration load at top of specimen. Crack propagation initiated from cover zone to core. Column failed due to concrete failure.

(a) Unheated Column

(b) Heated Column

Figure 5.7: Failure of M25(PCC) Mix Column

Reinforced Column(M25-FRC)

Column with M25(FRC) in unheated condition failed from due to buckling of reinforcement at top of column. Failure has been due to large amount of force concentration at top. It can be observed that core is intact, but the cover is damaged. As presented in Figure 5.8.

Heated column failed due to heavy concentration load at top of specimen. Concrete gets crushed and became incapable of bearing load.

(a) Unheated Column (b) Heated Column

Figure 5.8: Failure of M25(FRC) Mix Column

Reinforced Column(M60-PCC)

Column with M60(PCC) in unheated condition failed due to buckling of reinforcement at top of column. It can be observed that core was intact, but the cover is damaged.As presented in Figure 5.9.

Heated column failed due to heavy cover spalling from each face of column. Column load bearing surface area was reduced and column failed to reinforcement buckling.

(a) Unheated Column (b) Heated Column

Figure 5.9: Failure of M60(PCC) Mix Column

Reinforced Column(M60-FRC)

Column with M60(FRC) in unheated condition failed from top due to buckling of reinforcement at top of column.

Heated column failed due to heavy cover spalling from each face of column. Column load bearing surface area was reduced and column failed due to reinforcement buckling.As presented in Figure 5.10.

(a) Unheated Column (b) Heated Column

Figure 5.10: Failure of M60(FRC) Mix Column

Chapter 6

Concluding Remarks & Future Scope of Work

6.1 Summary

The present investigation incorporates comparison of four types of concrete mixes exposed to different elevated temperatures. Different concrete mixes include M25(PCC), M25(FRC), M60(PCC) & M60(FRC), respectively. Different Temperatures include 300°C, 500°C, 700°C & 900°C, respectively. The study includes plain concrete elements & reinforced concrete columns, respectively. The mechanical & physical properties of PCC & RC elements subjected to different elevated temperatures have been evaluated. Comparison has been done between four mixes i.e. namely M25(PCC), M25(FRC), M60(PCC) & M60(FRC). Hooked ended steel fibres have been incorporated in fibre reinforced concrete mixes namely M25(FRC) & M60(FRC) respectively. Mechanical properties for plain concrete elements included in this investigation are compressive strength, split tensile strength, flexural strength, modulus of elasticity & bond strength respectively. Physical properties for plain concrete elements included are weight loss, spalling effect & crack propagation. Additionally, Plain concrete specimens from four mixes are prepared and cast as per the mix design proportion which include M25(PCC), M25(FRC), M60(PCC) & M60(FRC). Specimens are water cured for 28 days. After curing period, specimens have been subjected to different elevated temperatures inside gas fired furnace for duration of 1 hour at target temperature. Specimens then allowed to cool at room temperature for 24 hours to achieve steady state condition. On later stage, physical properties and mechanical properties have been evaluated respectively. Average results of three specimens have been considered as a final result.

An attempt has been made to study behaviour of columns subjected to extreme elevated temperature i.e. 900°C. Design of RC columns with conventional HYSD reinforcement has been worked out using codal provisions. RC columns having dimensions 150mm×150mm×1000 mm have been cast with equal amount of reinforcement. Testing of columns have been carried out using axial compressive load at loading frame. The experimental results such as ultimate failure load, displacement, axial stress-lateral strain are measured. Also, crack patterns and failure modes for each concrete mixes have been studied. Three comparisons have been made, i.e. experimental ultimate failure load to that of theoretical failure load, ultimate failure load of heated specimen to that of unheated specimen, ultimate failure load of fibre reinforced columns[M25(FRC) & M60(FRC)] to that of Plain concrete columns[M25(PCC) & M60(PCC)], respectively. Average results of two columns have been considered as a final result.

6.2 Concluding Remarks

Following Concluding remarks have been made on basis of the work conducted in major project:

- Steel fibre incorporation in M25(FRC) & M60(FRC) mixes is found to be enhancing mechanical properties as compared to M25(PCC) & M60(PCC) in unheated condition, respectively. Physical deterioration of M25(PCC) and M25(FRC) mixes in terms of spalling is significantly lower exposed to 300°C & 500°C, respectively. For 700°C & 900°C physical deterioration in M25(FRC) is higher as compared to M25(PCC) in terms of spalling. Higher spalling in M25(FRC) may be due to the expansion of steel fibres which cause debonding of steel fibres from concrete paste. Physical deterioration in terms of spalling and weight loss for M60(PCC) and M60(FRC) mixes are higher as compared to M25(PCC) & M25(FRC) for each temperature range, respectively. Visual observation shows that, huge amount of surface spalling and corner spalling occurs in M60(PCC) & M60(FRC) exposed to 700°C & 900°C, which result into reduction in surface area.

- Compressive strength of M25(FRC) & M60(FRC) mixes in unheated condition are found to be at par with M25(PCC) & M60(PCC) mixes, respectively. Percentage loss in compressive strength for M25(PCC) is found to be higher as compared to M25(FRC) for each temperature range, respectively. Same phenomena is observed for M60 grade mixes, that is percentage loss in compressive strength for M60(PCC) mix is higher as compared to M60(FRC) mix for each temperature range, respectively. Additionally test results present that, percentage loss in compressive strength for M60(PCC) & M60(FRC) are significantly high for each temperature range as compared to M25(PCC) & M25(FRC), respectively.

- Plain concrete [M25(PCC) & M60(PCC)] is weak in tension. Incorporation of steel fibre in M25(FRC) & M60(FRC) mixes in unheated condition is found to be increasing split tensile strength significantly as compared to M25(PCC) & M60(PCC), respectively. Test results present that percentage loss in split tensile strength M25(PCC) is found to be very high as compared to M25(FRC) for each temperature range, respectively. Percentage loss in split tensile strength for M60(PCC) is noticeably high as compared to M60(FRC). Degradation in split tensile strength for M60(PCC) & M60(FRC) are significantly greater as compared to M25(PCC) & M25(FRC) for each temperature range, respectively. Destructive test results show that addition of steel fibre in M25(FRC) & M60(FRC) avoids sudden failure.

- Addition of steel fibre in M25(FRC) & M60(FRC) enhances flexural strength significantly in unheated condition as compared to that of M25(PCC) & M60(PCC) mixes, respectively. Destructive failure of M25(PCC) & M60(PCC) mixes is sudden in unheated & heated conditions. While destructive failure of M25(FRC) & M60(PCC) mixes is gradual in unheated & heated conditions. Percentage loss in flexural strength for M25(PCC) is found to be high as compared to M25(FRC) for each temperature range, respectively. Percentage loss in flexural strength for M60(PCC) is found to be extremely high as compared to M60(FRC) for each temperature range, respectively. Degradation in split tensile strength for M60(PCC) & M60(FRC) are significantly greater as compared to M25(PCC) & M25(FRC) for each temperature range, respectively.

- Modulus of Elasticity of M25(FRC) & M60(FRC) is found to be greater in unheated condition as compared to M25(PCC) & M60(PCC) respectively. Percentage loss in modulus of elasticity for M25(PCC) is found to be high as compared to M25(FRC) for each temperature range, respectively. Percentage loss in modulus of elasticity for M60(PCC) is found to be high as compared to M60(FRC) for each temperature range, respectively. Degradation in modulus of elasticity for M60(PCC) & M60(FRC) are significantly greater as compared to M25(PCC) & M25(FRC) for each temperature range, respectively.

- Experimental result shows that bond strength between concrete & steel bar reduces as temperature increase valid for each mix. Reason behind reduction in bond strength may be loosening of steel bar exposed to elevated temperatures which adversely affect the bond region. Result shows that bond strength of M25(FRC) & M60(FRC) are at par as compared to M25(PCC) & M60(PCC) in unheated condition, respectively. Percentage reduction in bond strength for M25(FRC) & M60(FRC) found to be at par as compared to M25(PCC) & M60(PCC) for each temperature range, respectively.

- Experimental investigation on RC columns demonstrate that, average ultimate load carrying capacity of heated (at 900°C) columns reduce as compare to that of unheated columns valid for each mix. Percentage reduction in average ultimate failure load of heated columns for M25(PCC), M25(FRC), M60(PCC) & M60(FRC) are 23.78%, 18.33%, 17.89% & 16.30%, respectively. Test result shows that M25(FRC) & M60(FRC) Columns have higher load carrying capacity in unheated condition as compared to M25(PCC) & M60(PCC) with equal amount of reinforcement provision, respectively. Percentage difference in ultimate load carrying capacity of FRC mix columns to that of PCC mix columns for M25(Unheated), M25(Heated), M60(Unheated) & M60(Heated) are +6.58%, +14.21%, 9.76% & 11.88% respectively.

- The average ultimate deflection of M25(FRC) Columns in Unheated and heated condition are 175% & 191% higher as compared to that of M25(PCC) Columns respectively. The average ultimate deflection of M60(FRC) Columns in Unheated and heated condition are 160% & 150% higher as compared to that of M60(PCC) Columns respectively. Reason behind more deflection and higher ultimate load carrying capacity of M25(FRC) & M60(FRC) columns may be that, fibres prevent early spalling of concrete which leads to significantly higher load carrying capacity of column and higher ultimate deflection.

- Destructive failure pattern demonstrate that failure behaviour of M25(PCC) & M60(PCC) mix columns are sudden, while for M25(FRC) & M60(FRC) mix columns gradual failure have been observed for unheated as well as heated condition. Experimental results present that displacement ductility in M25(FRC) & M60(FRC) mix columns is found to be significantly higher as compared to M25(PCC) & M60(PCC) in unheated as well as heated condition respectively.

- The above results shows that degradation in mechanical properties of M25(FRC) & M60(FRC) mixes are significantly lower as compared to M25(PCC) & M60(PCC) mixes respectively for each temperature range. M60(PCC) & M60(FRC) concrete mixes are more vulnerable to damage at elevated temperatures as compared to M25(PCC) & M25(FRC) mixes, respectively. Failure Pattern of heated M25(PCC) & M60(PCC) specimens are sudden while failure pattern of M25(FRC) & M60(FRC) specimens are gradual for each temperature range, respectively. Experimental result shows that addition of steel fibres in M25(FRC) & M60(FRC) Columns improve ultimate load carrying capacity in minor amount in unheated condition as compared to M25(PCC) & M60(PCC) columns, respectively. Ultimate load carrying capacity of M25(FRC) & M60(FRC) columns are noticeably high in heated condition as compared to M25(PCC) & M60(PCC) columns, respectively. Ultimate deflection and axial strain for M25(FRC) & M60(FRC) mix columns are significantly high in Unheated as well as heated condition as compared to M25(PCC) & M60(PCC) columns respectively. Steel fibre incorporation in columns are found to be enhancing displacement ductility in unheated as well as heated conditions respectively. M25(FRC) & M60(FRC) columns undergoes large deformation before failure in unheated as well as heated conditions, respectively.

6.3 Recommendation for Future Work

The study may be further extended to include following aspects in the work.

- Investigation can be further extended by incorporating polypropylene fibres & hybrid fibres in concrete mixes of different grades.
- Flexural behaviour of RC beams exposed to various elevated temperatures ranging between 300°C to 900°C can be evaluated.
- Exposure duration at target temperature may be further extended (more than 1 hour) to understand fire severity.
- RC columns with different amount of cover provisions may be tested.

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