MECHANICAL PROPERTIES OF HIGH VOLUME FLY ASH (HVFA) CONCRETE SUBJECTED TO ELEVATED TEMPERATURES UP TO 120˚C

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CERTIFICATE

This is to certify that the thesis entitled '**MECHANICAL PROPERTIES OF HIGH VOLUME FLY ASH (HVFA) CONCRETE SUBJECTED TO ELEVATED TEMPERATURES UP TO 120˚C**', submitted by Ms. **Inderpreet Kaur** in partial fulfillment of requirements, for the award of degree of Master of engineering in Civil (structures) Engineering of Thapar Institute of Engineering and Technology (Deemed University), Patiala, is a bonafide work carried out by him under my supervision and guidance.

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I will be failing in my duties if I do not acknowledge my sense of gratitude to God Almighty, my Father and Mother, whose blessing have made me reach my destination.

Place: Patiala **(*Inderpreet Kaur*)**

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ABSTRACT

Waste disposal is one of the major problems being faced by all the nations across the globe. Fly ash is an industrial by-product, generated from combustion of coal in the thermal power plants. The increasing scarcity of raw materials and an urgent need to protect the environment against pollution has accentuated the significance of developing new building materials based on industrial waste generated from coal fired thermal power station and is creating unmanageable disposal problems due to its potential to pollute the environment.

Pozzolanic concretes are used extensively throughout the world where oil, gas, nuclear and power industries are among the major users. The applications of such concretes are increasing day by day due to their superior structural performance, environmental friendliness, and energy conserving implications. Apart from the usual risk of fire, these concretes are exposed to high temperatures for considerable periods of time in the above-mentioned industries. Although concrete is generally believed to be an excellent fireproofing material, but there is extensive damage or even catastrophic failure at high temperatures. At high temperatures, chemical transformation of the gel weakened the matrix bonding, which brought about a loss of strength of fly ash concrete.

Fly ash is used as a mineral addition in concrete to improve its strength and durability characteristics. Fly ash can be used either as an admixture or as a partial replacement of cement or as a partial replacement of fine aggregates or total replacement of fine aggregate and as supplementary addition to achieve different properties of concrete.

In the present study, the compressive strength, split tensile strength and modulus of elasticity of fly ash concrete at elevated temperature up to 120˚C with mix proportions of 1:1.45:2.2:1.103 with a water cement ratio of 0.5 by weight was determined. Cement was replaced with three percentages of fly ash. The percentages of replacements were 30, 40 and 50 % by weight of cement. Tests were performed for compressive strength, split tensile strength and modulus of elasticity. Compressive strength, split tensile strength and

modulus of elasticity were performed at room temperature, 80˚C, 100˚C, and 120˚C for all types of fly ash concrete at different curing periods (28 and 56 days). Reference concrete without fly ash has also been used.

Test results showed that the compressive strength, split tensile strength and modulus of elasticity of concrete having cement replacement up to 30% was comparable to the reference concrete without fly ash. Compressive strength, split tensile strength and modulus of elasticity of concrete mixtures with 30%, 40% and 50 % of fly ash as cement replacement was lower than the control mixture at all ages and that the strength of all mixtures continued to increase with the age. With the increase in temperature, compressive strength of concrete mixes with 30%, 40% and 50 % of fly ash as cement replacement decreases by 11.4%, 30.1%, 28.9%, and 27.5% at 120˚C when compared to room temperature.

CHAPTER-1

INTRODUCTION

1.1 General

Pozzolanic concretes are used extensively throughout the world where oil, gas, nuclear and power industries are among the major users. The applications of such concretes are increasing day by day due to their superior structural performance, environmental friendliness, and energy conserving implications. Apart from the usual risk of fire, these concretes are exposed to high temperatures for considerable periods of time in the above-mentioned industries. Although concrete is generally believed to be an excellent fireproofing material, but there is extensive damage or even catastrophic failure at high temperatures. At high temperatures, chemical transformation of the gel weakened the matrix bonding, which brought about a loss of strength of fly ash concrete.

The search for alternative binders, or cement replacement materials, has been carried out for decades. Research has been conducted on the use of fly ash, volcanic ash, volcanic pumice, pulverized-fuel ash, blast slag and silica fume as cement replacement material. Fly ash and others are pozzolanic materials because of their reaction with lime liberated during the hydration of cement. These materials can also improve the durability of concrete and the rate of gain in strength and can also reduce the rate of liberation of heat, which is beneficial for mass concrete.

Concretes containing mineral admixtures are used extensively throughout the world for their good performance and for ecological and economic reason. The effect of high temperature on concrete containing fly ash or natural pozzolans has not been investigated in detail unlike fly ash concrete that has been under investigation since the 1960s. There are changes in the properties of concretes, particularly in the range of 100–300 °C. Above 300°C, there is decrease in mechanical characteristics. However, there is a decrease in strength due to the variety of high temperature condition tested, and the variety of constituent materials of concrete used. The behavior of concrete subjected to high temperatures is a result of many factors; such as heating rate, peak temperatures, dehydration of

C–S–H gel, phase transformations, and thermal incompatibility between aggregates and cement paste. On the other hand, quality control of concrete, by means of non-destructive methods, in structures subjected to fire or not so high temperature exposure conditions, is not particularly easy to be carried out. The correlation already exists usually refers to the hydration age of 28 days.

High temperatures induce severe micro-structural changes that alter mechanical properties of portland cement concrete. The pore structure and thought its physical properties change with time following the hydration and aging processes and they are strongly influenced not only by the mechanical load but also by the thermal-hygrometric state of concrete and their time history.

The physical and chemical changes in concrete under high temperatures depend not only on the matrix composition but also on the type of aggregate (mineralogical characteristics, dilatation etc.). Other factors that have influence are the water/cement ratio, the porosity, humidity and age of concrete. As the cement paste is exposed to increasing temperatures the following effects can be distinguished: the expulsion of evaporable water (100°C), the beginning of the dehydratation of the hydrate calcium silicate (180°C), the decomposition of calcium hydroxide (500°C) and of the hydrate calcium silicate (that begins around 700°C). The alterations produced by high temperatures are more evident when the temperature surpasses 500°C. At this temperature level, most changes experienced by concrete can be considered irreversible.

At the structural level, the behavior of concrete elements exposed to high temperatures is characterized spalling, that is a brittle failure with most cracks parallel to the heated surface. The mechanical properties of concrete in general are adversely affected by thermal exposure. However, the effects of thermal exposure on the mechanical properties of high performance concrete (HPC) have found to be more pronounced than the effects on conventional concrete. When HPC exposed to relatively rapid heating (above l°C/min), it has been shown to be more prone to dramatic spalling failure. The spalling failures in laboratory conditions have been characterized from being progressive (continuous spalling of small scales on the specimen's surface when subjected to radiant heating) to explosive (sudden disintegration of the specimen accompanied by the release of a large amount of energy which projects the broken concrete fragments in all directions with high velocity). It has been theorized that the higher susceptibility of HPC to explosive spalling at high temperature is due, in part, to its lower permeability, which limits the ability of water vapor to escape from the pores. This results in a build-up of pore pressure within the concrete. As heating increases, the pore pressure also increases. This increase in vapor pressure continues until the internal stresses become so large as to result in sudden, explosive spalling.. Often, explosive spalling has occurred to only a few HPC specimens from a larger group of specimens that were subjected to identical testing conditions. This erratic behavior makes it difficult to predict with certainty under what conditions HPC will fail by explosive spalling.

1.2 Effect of fire on concrete

The human civilization started from the time when we learned how to make use of fire. However, fire also brings disasters to humans at any time and at any place when it is out of control. It was found that majority of fire events took place in buildings, and they caused the greatest personal and economic losses. In Hong Kong, most of the building structures were made of reinforced concrete. Although concrete is a non-combustible material, it is not immune from fire attacks. One of the unforgettable fire events here is the Garley reinforced concrete building fire happened on 20 November 1996. This fire killed 40 people and injured over 80 others. The building suffered from different degrees of fire-damages, and is still unoccupied at this moment.

In Hong Kong, the requirements of fire resisting construction are mandated in Part XV of the Building (Construction) Regulations, focusing on the resistance to the spread of fire/smoke and the maintenance of the integrity of structural elements of buildings during a fire attack. Guidance on how these requirements may be fulfilled in designing a building structure is elaborated in the Code of Practice for Fire Resisting Construction (Building Authority, 1996) and the PNAP 204: Guide to Fire Engineering Approach (Buildings Department, 1998). In addition, there are two other codified publications on the provisions of means of access for firefighting and escapes in case of a fire (Building Authority, 1995, 1996). However, a local code of practice on how to quantify the strength and serviceability of a building structure after a fire is still not available. Consequently, a structural engineer has to rely on his/her own knowledge on the properties of the structural materials during and after a fire, and to quantify the aforesaid properties to appraise the structural performance of a fire-damaged building.

1.2.1 Damage Mechanisms of Concrete under Fire

The effects of high temperatures on high strength concrete (HSC) materials have also been studied since the past decade. Although there are significant differences between normal and high strength concretes in fire performance, their thermal damages (crack formation, explosive spalling, and degradation of mechanical/durability properties) are similar and mainly arise from (i) thermal mismatch, (ii) decomposition of hydrates, and (iii) pore pressure.

1.2.1.1 Thermal mismatch

When a concrete member is exposed to fire, the temperature gradient across the depth of the section will be built up. The thermal expansion of the outer layer of concrete under a high temperature is partially restrained by the inner layer of concrete under a lower elevated temperature, and this generates tensile thermal stresses in the concrete. The steeper the temperature gradient, the higher will be the thermal stresses.

Another situation is that even a concrete member is subjected to an ideal uniform temperature field across its thickness, thermal stresses can still be developed because of the incompatibility of the coefficients of thermal expansion of the constitutive materials in concrete. Concrete is a brittle composite material that consists of binder (cement) paste and aggregates (fine and coarse aggregates). These materials have different mechanical and physical properties, including different coefficients of thermal expansion. At a lower elevated temperature, say 100°C, the thermal expansion of the cement paste is slightly greater than that of the granite. Consequently, in the concrete matrix, the cement paste is under hydrostatic compression, and the granite aggregates are under biaxial compression and tension. As the temperature further increases, for example over 400°C, the thermal strain of the cement paste changes to negative (shrinking) due to chemical changes, whereas the granite continues to expand. The corresponding stresses in concrete are that the granite aggregates are under hydrostatic compression and the cement paste is under biaxial compression and tension.

1.2.1.2 Decomposition of hydrates

The mechanical properties of concrete depend largely on the hydration products (calcium silicate hydrate gel, calcium hydroxide, and ettringite) formed during the hydration reaction between the cementituous constituents and water. When concrete is exposed to a fire attack, free water in the concrete matrix will firstly be removed through a physical process such as evaporation at a lower elevated temperature. As the temperature further increases, disintegration of hydrates and loss of chemically bonded water will take place.

The decomposition of calcium hydroxide occurred at about 350°C, and partial volatilization of calcium silicate hydrate gel commenced at about 500°C. The pore size and porosity of the hydrate matrix will increase, and the mechanical properties (strength and elastic modulus) of the hydrates will be weakened. Moreover, at 573°C, the crystal structure of quartz in a siliceous aggregate transforms from a low temperature α-phase to a high temperature β-phase. Such transformation is accompanied by an approximately one percent volume increase, which accelerates the disintegration process of the hydrates. All these changes make the mechanical properties of a heated concrete (in a macro-scale) be temperature-dependent.

1.2.1.3 Pore pressure

The pore pressure developed in a heated concrete is derived from the evaporation of water within the porous media (free water) and from the decomposition of C-S-H gel and calcium hydroxide (chemically combined water). The highest pore pressure occurred between 220°C and 240°C in high strength concrete (HSC) and between 190°C and 210°C in normal strength concrete (NSC). The magnitude of the pore pressure depends on (i) the moisture level (degree of saturation), (ii) the permeability of concrete, and (iii) the heating rate. The maximum pore pressure could reach 3 MPa and 1 MPa in unsealed heated HSC and NSC respectively. Low permeability and dense microstructure of HSC are probably the causes for creating high pore pressure that has been considered as a key factor for explosive spalling of concrete. The concrete with compressive strength of less than 60 MPa, no spallling would take place even though the concrete was fully saturated. For the concrete with compressive strength of more than 60 MPa, the chance of explosive spalling occurrence increases with moisture content. However, it is still difficult to quantify with certainty under what conditions HSC would fail in an explosive manner.

1.3 Fly Ash

Fly ash is a finely divided residue resulting from the combustion of pulverized coal and transported by the flue gases of boilers fired by pulverized coal. It is available in large quantities in the country, as a waste product, from a number of thermal power stations and industrial plants using pulverized coal as fuel from boilers. Its availability is likely to increase with the increased industrialization in the country. The use of fly ash as a pozzolana and a fine aggregate, also for other allied purposes is well established in a number of countries abroad, but it has come in vogue in India only recently. Some recent investigations on Indian fly ashes have proved their suitability for various uses. Indigenous fly ashes for replacement of cement and for use with lime, as an admixture for cement, concrete bituminous mixtures and as fine aggregate for mortar and concrete have already been successfully tried out and greater attention is now being paid to fully exploit the potentialities of fly ash as a construction material.

The quantity of fly ash produced depends upon several factors and of which main ones are-

1. The quantity of coal consumed.
2. The quality of coal.
3. The type of pulverization.
4. The type of boiler for combustion.
5. The type of collection system, varying from mechanical to electrical precipitators or bag house and fabric filters.

Fly ash resulting from the combustion of pulverized coal in boiler of thermal plant is grey in colour and alkaline in nature. The particle size may correspond to that of silty sand to silty clay i.e. between 5-120 microns. The flyash also contains heavy metals depending upon the coal burnt. This is prevented from polluting atmosphere by extracting it from the flue gases by ESP's (Electro- static- precipitators). At thermal plants at Ropar, Bathinda and Lehra Mohabat, the fly ash collected in the ash hopper is removed from boilers house by wet as well as dry system. In the dry system the ash collected in silos where from it is available for various uses. In the wet system, fly ash is mixed with water in a mixing sump to form ash slurry and is pumped into ash ponds. Table 1.1 present chemical and physical requirements for fly ash and natural pozzolans for use as a mineral admixture in Portland cement concrete and Table 1.2 present chemical requirement of fly ash.

1.3.1 Classification of Fly Ash

ASTM – C 618-93 categorizes natural pozzolans and fly ashes into the following three categories: -

1. **Class N Fly ash:** Raw or calcined natural pozzolans such as some diatomaceous earths, opaline chert and shale, stuffs, volcanic ashes and pumice come in this category. Calcined kaolin clay and laterite shale also fall in this category of pozzolans.
2. **Class F Fly ash:** Fly ash normally produced from burning anthracite or bituminous coal falls in this category. This class of fly ash exhibits pozzolanic property but rarely if any, self-hardening property.
3. **Class C Fly ash:** Fly ash normally produced from lignite or sub- bituminous coal is the only material included in this category. This class of fly ash has both pozzolanic and varying degree of self cementitious properties. (Most class C fly ashes contain more than 15 % CaO. But some class C fly ashes may contain as little as 10 % CaO.

1.3.2 Physical Properties of Fly Ash

Particle Morphology

As per morphological studies, fly ash particles usually consist of clear glassy spheres and spongy aggregate ranging in diameter from 1 to 150 μm, the majority being less than 45 μm as seen under energy dispersive X-ray analysis (EDXA).

Fineness

Fineness is one of the primary physical characteristics of fly ash that relates to its pozzolanic activity. A large fraction of ash particles is smaller than 3 μm in size. In bituminous ashes, the particles sizes range from less than 1 to over 100 μm.

Specific gravity

The specific gravity of fly ash is related to shape as well as chemical composition of particles. Specific gravity of fly ash usually varies from 1.3 to 4.8. Coal particles with some minerallic impurities have specific gravity in between 1.3 to 1.6. Opaque spherical magnetite (ferrite spinal) and hematite particles, light brown to black in colour, when present in sufficient quantity in fly ash increases the specific gravity to about 3.6 to 4.8.

1.3.3 Chemical Properties of Fly Ash

Chemical constituents in fly ash reported in terms of oxides include silica (SiO_2), alumina (Al_2O_3), calcium oxides (CaO), iron oxide (Fe_2O_3), magnesium oxide (MgO), and oxides of tiatanium (TiO_2), sulphur (SO_3), sodium (Na_2O), and potassium (K_2O). Unburned carbon is another major constituent in all the ashes. Amongst these SiO_2 and Al_2O_3 together make up about 45 to 80 % of the total ash. The sub-bituminous and lignite coal ashes have relatively higher proportion of CaO and MgO and lesser Proportions of SiO_2, Al_2O_3, and Fe_2O_3 as compared to the bituminous coal ashes.

Total oxides

For class F fly ashes, the sum of silica, alumina and iron oxide (SiO_2 + Al_2O_3 + Fe_2O_3), must be at least 70 %, whereas for class C the required minimum is 50 %. For fly ash to act as a pozzolan, it is necessary to have chemical constituents capable of reacting with lime in the presence of water.

Sulphur trioxide (SO_3)

The maximum SO_3 content allowed in the fly ash by ASTM C 618 is 5.0 %. The SO_3 content has been reported to affect to some degree the early age compressive strength of mortar and concrete specimens. The higher the SO_3 content, the higher is the resultant strengths.

Moisture

A 3.0 % limit on moisture content is specified in ASTM C 618 to prevent caking and packing of the fly ash during shipping and storage and to control uniformity of fly ash shipments. Any amount of moisture in Class C fly ash will cause hardening from hydration of its cementitious compounds.

Carbon content (Loss on ignition)

The maximum permissible loss on ignition is related to the amount of carbon or unburnt coal in the fly ash. For some of the fly ashes, there is a significant difference in carbon content and Loss on Ignition (LOI) yet the specification restricts the LOI and not the unburnt carbon. The present ASTM C 618 sets, the limit on loss on ignition at 6.0 % for Class F and Class C fly ashes.

1.3.4 Mineralogical Characteristics

An X- ray diffraction study of the crystalline and glassy phases of a fly ash is commonly called mineralogical analysis of fly ash. Fly ashes, generally, have 15 to 45 % crystalline matter. The high calcium ashes, derived from low- rank sub – bitumininous and lignite coals, contain larger amounts of crystalline matter ranging between 25 – 45 %. Although high calcium Class C ashes may have somewhat less glassy or amorphous material, they also have certain crystalline phases such as anhydride ($CaSO_4$), tricalcium aluminate ($3CaOAl_2O_3$), calcium sulpo – aluminate $CaSAl_2O_3$ and very small amount of free lime (CaO) that participate in producing cementitious compounds.

Anhydride ($CaSO_4$)

It forms from reaction of CaO, SO_2 and O_2 in the furnace or flue. The amount of anhydrite increases with the increasing SO_3 and CaO contents in the ash. Anhydrite is a characteristics phase in high calcium class-C fly ashes. For most ashes, only about half of the SO_3 is present as anhydrite.

Periclase (MgO)

Periclase refers the crystalline form of magnesium oxide (MgO). It is always present in high calcium ash and commonly found in intermediate calcium ash. This form of MgO present in fly ash affects the soundness of the resulting concrete through its expansive hydration to brucite, Mg (OH_2).

Magnetite and Hematite

There is at least a small amount from 0.1 to 1 % of iron present as hematite in almost all type of flyashes. High calcium Class C flyashes have however less amount of hematite as well as total Fe_2O_3.

Tricalcium Aluminate (3 CaO, Al_2O_3)

High calcium class-C flyash invariably contain tri-calcium aluminate with its relative content increasing with an increase of CaO content of the ash. Sometimes intermediate calcium ashes with CaO content of 8 to 15 %, have also been found to contain this compound.

Table 1.1 Requirements for fly ash and natural pozzolans for use as a mineral admixture in Portland cement concrete as per ASTM C 618-93.

Requirements	**Fly Ash Classification**		
	N	**F**	**C**
Chemical Requirements			
SiO2 + Al2O3 + Fe2O3, min %	70.0	70.0	50.0
SO3, max %	4.0	5.0	5.0
Moisture content, max %	3.0	3.0	3.0
Loss on ignition, max %	10.0	6.0	6.0

Physical Requirements			
Amount retained when wet sieved on 45 µm. Sieve, max %	34	34	34
Pozzolanic activity index, with Portland cement at 28 days, min % of control	75	75	75
Pozzolanic activity index, with lime, at 7 days, min (MPa)	5.5	5.5	-
Water requirement, max % of control	115	105	105
Autoclave expansion or contraction, max %	0.8	0.8	0.8
Specific gravity, max variation from average.	5	5	5
Percentage retained on 45 sieve, max variation, and percentage points from average.	5	5	5

Table 1.2 Chemical Requirements

S. No.	Characteristics	Requirement (%)
1.	Silicon dioxide (SiO_2) + aluminium oxide (Al_2O_3) + iron oxide (Fe_2O_3), percent by mass, Min.	70.00
2.	Silicon dioxide (SiO_2), percent by mass, Min.	35.00
3.	Magnesium oxide (MgO), percent by mass, Max.	5.00
4.	Total sulphur as sulphur trioxide (SO_3), percent by mass, Max.	2.75
5.	Available alkalis as sodium oxide (Na_2O), percent by mass, Max.	1.5
6.	Loss on Ignition, percent by mass, Max.	12.0
7.	Moisture content, percent by mass	3.0

1.3 High-Volume Fly Ash Concrete

Fly ash, a principal by-product of the coal-fired power plants, is well accepted as a pozzolanic material that may be used either as a component of blended Portland cements or as a mineral admixture in concrete. In commercial practice, the dosage of fly ash is limited to 15%-20% by mass of the total cementitious material. Usually, this amount has a beneficial effect on the workability and cost economy of concrete but it may not be enough to sufficiently improve the durability to sulfate attack, alkali-silica expansion, and thermal cracking. For this purpose, larger amounts of fly ash, on the order of 25%-35% are being used.

From theoretical considerations and practical experience the authors have determined that, with 50% or more cement replacement by fly ash, it is possible to produce sustainable, high-performance concrete mixtures that show high workability, high ultimate strength, and high durability.

1.4.1 Fly ash as a water reducer

Too much mixing-water is probably the most important cause for many problems that are encountered with concrete mixtures. There are two reasons why typical concrete mixtures contain too much mixing-water. Firstly, the water demand and workability are influenced greatly by particle size distribution, particle packing effect, and voids present in the solid system. Typical concrete mixtures do not have an optimum particle size distribution, and this accounts for the undesirably high water requirement to achieve certain workability. Secondly, to plasticize a cement paste for achieving a satisfactory consistency, much larger amounts of water than necessary for the hydration of cement have to be used because Portland cement particles, due to the presence of electric charge on the surface, tend to form flocs that trap volumes of the mixing water. It is generally observed that a partial substitution of Portland cement by fly ash in a mortar or concrete mixture reduces that water requirement for obtaining a given consistency. With HVFA concrete mixtures, depending on the quality of fly ash and the amount of cement replaced, up to 20% reduction in water requirements can be achieved. This means that good fly ash can act as a superplasticizing admixture when used in high-volume. The phenomenon is attributable to three mechanisms. First, fine particles of fly ash get absorbed on the oppositely charged surfaces of cement particles and prevent them from flocculation. The cement particles are thus effectively dispersed and will trap large amounts of water, that means that the system will have a reduced water requirement to achieve a given consistency. Secondly, the spherical shape and the smooth surface of fly ash particles help to reduce the inter-particle friction and thus facilitate mobility. Thirdly, the “particle packing effect” is also responsible for the reduced water demand in plasticizing the system. It may be noted that both Portland cement and fly ash contribute particles that are mostly in the 1-to 45-µm size ranges, and therefore serve as excellent fillers for the void space within the aggregate mixture. In fact, due to its lower density and higher volume per unit mass, fly ash is a more efficient void-filler than Portland cement.

1.4.2 Drying shrinkage

Perhaps the greatest disadvantage associated with the use of neat Portland cement concrete is cracking due to drying shrinkage. The drying shrinkage of concrete is directly influenced by the amount and the quality of the cement paste present. It increases with an increase in the cement paste-to-aggregate ratio in the concrete mixture, and also increases with the water content of the paste. Clearly, the water-reducing property of fly ash can be advantageously used for achieving a considerable reduction in the drying shrinkage of concrete mixtures. Due to a significant reduction in the water requirement, the total volume of the cement paste in the HVFA concrete is only 25% as compared to 29.6% for the conventional Portland-cement concrete, which represents a 30% reduction in the cement paste-to aggregate volume ratio.

1.4.3 Thermal cracking

Thermal cracking is of serious concern in massive concrete structures. It is generally assumed that this is not a problem with reinforced-concrete structures of moderate thickness, e.g. 50-cm thick or less. However, due to the high reactivity of modern cements cases of thermal cracking are reported even from moderate-size structures made with concrete mixtures of high-cement content that tend to develop excessive heat during curing. The physical-chemical characteristics of ordinary Portland cements today are such that very high heat-of-hydration is produced at an early age compared with that of normal Portland cements available 40 years ago. Also, high early strength requirements in modern construction practice are usually satisfied by an increase in the cement content of the concrete mixture. Further, there is considerable construction activity now in the hot-arid areas of the world where concrete temperatures in excess of 60ºC are not uncommon within a few days of concrete placement.

For unreinforced mass-concrete construction, several methods are employed to prevent thermal cracking, and some of these techniques can be successfully used for mitigation of thermal cracks in massive reinforced-concrete structures. For instance, a 40-MPa concrete mixture containing 350 kg/m3 Portland cement can raise the temperature of concrete by approximately 55-60ºC within a week if there is no heat loss to the environment. However, with a HVFA concrete mixture containing 50% cement replacement with a Class F fly ash, the adiabatic temperature rise is expected to be 30-35ºC. As a rule of thumb, the maximum temperature difference between the interior and exterior concrete should not exceed 25ºC to avoid thermal cracking. This is because higher temperature differentials are accomplished by rapid cooling rates that usually result in cracking. Evidently, in the case of conventional concrete it is easier to solve the problem either by keeping the concrete insulated and warm for a longer time in the forms until the temperature differential drops below 25ºC or by reducing the proportion of Portland cement in the binder by a considerable amount. The latter option can be exercised if the structural designer is willing to accept a slightly slower rate of strength development during the first 28 days, and the concrete strength specification is based on 90-day instead of 28-day strength.

1.4.4 Water-tightness and durability

In general, the resistance of a reinforced-concrete structure to corrosion, alkali aggregate expansion, sulfate and other forms of chemical attack depends on the water-tightness of the concrete. The water-tightness is greatly influenced by the amount of mixing-water, type and amount of supplementary cementing materials, curing, and cracking resistance of concrete. High-volume fly ash concrete mixtures, when properly cured, are able to provide excellent water-tightness and durability. When a concrete mixture is consolidated after placement, along with entrapped air, a part of the mixing-water is also released. As water has low density, it tends to travel to the surface of concrete. However, not all of this "bleed water" is able to find its way to the surface. Due to the wall effect of coarse aggregate particles, some of it accumulates in the vicinity of aggregate surfaces, causing a heterogeneous distribution of water in the system. Obviously, the interfacial transition zone between the aggregate and cement paste is the area with high water/cement and therefore with more available space that permits the formation of a highly porous hydration product containing large crystals of calcium hydroxide and ettringite. Microcracks due to stress are readily formed through this product because it is much weaker than the bulk cement paste with a lower water/cement. It has been suggested that microcracks in the interfacial transition zone play an important part in determining not only the mechanical properties but also the permeability and durability of concrete exposed to severe environmental conditions. This is because the rate of fluid transport in concrete is much larger by percolation through an interconnected network of microcracks than by diffusion or capillary suction. The heterogeneities in the microstructure of the hydrated Portland cement paste, especially the existence of large pores and large crystalline products in the transition zone, are greatly reduced by the introduction of fine particles of fly ash.

With the progress of the pozzolanic reaction, a gradual decrease occurs in both the size of the capillary pores and the crystalline hydration products in the transition zone, thereby reducing its thickness and eliminating the weak link in the concrete microstructure. In conclusion, a combination of particle packing effect, low water content, and pozzolanic reaction accounts for the eventual disappearance of the interfacial transition zone in HVFA concrete, and thus enables the development of a highly crack-resistant and durable product.

CHAPTER 2

LITERATURE REVIEW

> Cheng et al. [1] investigated on the effects of high temperature on the strength and stress-strain relationship of high strength concrete (HSC). Stress-strain curve tests were conducted at various temperatures (20, 100, 200, 400, 600, and 800ºC) for four types of HSC. Results from stress-strain curve tests show that plain HSC exhibits brittle properties below 600ºC, and ductility above 600ºC. HSC with steel fibers exhibits ductility for temperatures over 400ºC. The compressive strength of HSC decreases by about a quarter of its room temperature strength within the range of 100-400ºC. The strength further decreases with the increase of temperature and reaches about a quarter of its initial strength at 800ºC. the strain at peak loading increases with temperature, from 0.003 at room temperature to 0.02 at 800ºC.

Estakhri and Mohidekar [2] compiled data for 18 power plants located throughout Texas and determined that a total of 6.6 million tons of fly ash are produced annually in Texas and about 2.7 million tons (or 40%) are generally sold for use in concrete or other end products. Researchers estimated production of concrete in Texas and determined that if 60 percent of the portland cement used in Texas concrete production were replaced with fly ash, carbon dioxide emissions could potentially be reduced by 6.6 million tons annually by the year 2015. Fly ash can improve the workability and reduces the heat of hydration in fresh concrete. It can improve the strength, permeability, and resistance to chemical attack of the hardened concrete. Canadian research has led to the development of high volume fly ash (HVFA) concrete. In this concrete, up to 60 percent of the

Portland cement is replaced with Class F fly ash. HVFA concrete exhibits excellent strength, workability, and low temperature rise. One barrier perceived by industry is that it takes a longer time to set. This can, however, be a distinct advantage during a typical Texas summer.

Felicetti and Gambarova [3] reported on the use of high strength concretes (f_c>60MPa) in the special structures designed to work in a high temperature environment or to withstand severe thermal accidents requires the mechanical properties of the material to be assessed with regard to high temperature effects. The "residual" mechanical properties of two high strength concretes (f_c=72 and 95MPa), with siliceous aggregates (mostly flint) are studied under uniaxial compression after a single thermal cycle at 105, 250, 400, and 500ºC. The two concretes are very sensitive to high temperature, and- in spite of their good mechanical properties at room temperature (f_c>70 MPa), even with lean mix and a relatively high water/binder ratio- both the strength and the stiffness decrease dramatically at and beyond 250ºC: after a cycle at 250ºC the residual strength is down to 80 to 66 percent of the strength at room temperature, the degradation of the Young's modulus is even more impressive. Both concretes are very brittle below 250ºC; however, the higher the temperature of the thermal cycle, the softer the material becomes, but, owing to the simultaneous decrease of both the compressive strength and Young's modulus, the toughness does not improve significantly below 250ºC, but increases markedly beyond 400ºC. The failure mode of cylinders is characterized by splitting through both the mortar and the aggregate particles: no shear bands occurred at any time. However, below 250ºC only a few splitting cracks form (just prior to or at the peak stress), while beyond 250ºC the splitting cracks are more numerous and thinner, and the fracture process is more disordered owing to previous thermal fracturing.

Fu et al. [4] developed a 2-D mesoscopic thermoelastic damage model used to study the thermal stress field and associated fracture in a cement-based composite with multiple circular or irregular inclusions subjected to elevated temperatures. It is found that the thermal stress field and the associated cracking are dominated by (i) the thermal mismatch between the matrix and the inclusions, (ii) the arrangement of the inclusions, and (iii) the heterogeneity and the shape of the inclusions. Thermal radical cracks firstly occur between two adjacent inclusions, which have the shortest distance apart or sharp corners when the coefficient of thermal expansion of the inclusions is greater than that of the matrix. The propagation of radical cracks in the matrix will be terminated by the presence of an inclusion with higher strength at the crack tip. The numerical findings are also used to discuss the thermal cracking histories of concretes with different aggregate grading arrangements.

Ghosh and Nasser [5] reported on the effects of high temperatures (up to 232ºC) and high pressures (up to 13.8 MPa) on the strength and elasticity of concrete (50 to70 MPa) containing lignite fly ash and a fixed percentage of condensed silica fume. Five different temperatures varying from 21 to 232ºC and three different pressures varying from 5.2 to 13.8 MPa were used. A gradual deterioration of strength and static modulus of elasticity was observed with the rise in temperature. A physico-chemical transformation of the paste took place with the rise in temperatures. At high temperatures, chemical transformation of the gel weakened the matrix bonding, which brought about a loss of strength and elasticity of fly ash and silica fume concrete.

Hansen [6] investigate the effect of curing temperatures of 5, 25 and 50°C on sealed cured cement paste samples containing between 7.5 and 25 % pozzolanic filler in the form of fly ash and micro silica at a water/powder ratio of app. 0.23 has been studied. The results indicate that the main effect of elevating the curing temperature is densification or crystallization of the hydrates, liberating significant amounts of surface adsorbed water and increasing the amount of chemically bound water. This increase in available water counters the effect of reduced w_n/c ratio of the hydrates at higher temperature, and the retaining of water by physical binding on the surfaces of hydrates formed by pozzolanic reactions. Within the considered composition and curing temperature range the net result of these mechanisms ranges from an increase in chemically bound water content with increased curing temperature at low to moderate pozzolanic filler contents (especially micro silica contents appears significant) to a negligible effect at high filler contents.

> This increase in chemically bound water (and thus formed hydrates) due to the above mentioned mechanisms may be the cause for the increased compressive strengths at increased curing temperatures observed in some low water content systems.

Handoo et al. [7] reported the results of an experimental investigation dealing with concrete cubes prepared from ordinary Portland cement (OPC) of known chemical, mineralogical, and physical performance characteristics and fired to various temperature regimes up to 1000ºC in steps of 100ºC for a constant period of 5 h by using X ray diffraction (XRD) and DTA/TGA to establish the effect of elevated temperatures on the mineralogical changes occurring in the hydrated phases of concrete. The changes in physical state of concrete were studied by measuring ultrasonic pulse velocity (UPV) and consequent deterioration in the compressive strength with increase in temperature. Scanning electron microscopy (SEM) studies showed distinct morphological changes corresponding to deterioration of concrete exposed to higher temperatures. Reduction in the compressive strength of concrete exposed to beyond 500ºC is quite rapid. Complete decomposition of portlandite beyond 700ºC at the surface and beyond 900ºC at the core results in total deterioration of concrete. Morphological studies confirm clear deformation of well-developed calcium hydroxide crystals and C-S-H gel beyond 600ºC. The decrease in portlandite content and consequent reduction in compressive strength with increase in temperatures can be used for assessing the condition of building elements subjected to accidental fires.

Hossain and Lachemi [8] carried out an investigation on the strength and durability performance of concretes incorporating 0 to 40% of volcanic ash (VA) as cement replacement (by mass) subjected to high temperatures up to 800ºC. The strength properties were assessed by unstressed residual compressive strength, while durability was investigated by rapid chloride permeability test (RCPT), mercury intrusion porosimetry (MIP), differential scanning calorimetry (DSC) and crack pattern observations. Volcanic ash concrete (VAC) showed good performance showing higher residual strength, higher chloride resistance and higher resistance against deterioration particularly at temperatures below 600ºC compared with the control ordinary Portland cement (OPC) concrete. The improved performance of VAC can be attributed to the refinement of pore structure, lowering the presence of free chloride due to Friendel's salt formation and pozzolanic action due to the presence of VA. The deterioration of both strength and durability of VAC increased with the increase of temperature up to 800ºC due to substantial reduction in residual strength and an increase in pore volume and pore diameter.

Ichikawa and England [9] carried out investigations on prediction of moisture migration and pore pressure build-up in non-uniformly heated concrete is important for safe operation of concrete containment vessels in nuclear power reactors and for assessing the behavior of fire-exposed concrete structures. (1) Changes in moisture content distribution in a concrete containment vessel during long-term operation should be investigated, since the durability and radiation shielding ability of concrete are strongly influenced by its moisture content. (2) The pressure build-up in a concrete containment vessel in a postulated accident should be evaluated in order to determine whether a venting system is necessary between liner and concrete to relieve the pore pressure. (3) When concrete is subjected to rapid heating during a fire, the concrete can suffer from spalling due to pressure build-up in the concrete pores. It presents a mathematical and computational model for predicting changes in temperature, moisture content and pore pressure in concrete at elevated temperatures. A pair of differential equations for one-dimensional heat and moisture transfer in concrete are derived from the conservation of energy and mass, and take into account the temperature-dependent release of gel water and chemically bound water due to dehydration. These equations are numerically solved by the finite difference method. In the numerical analysis, the pressure, density and dynamic viscosity of water in the concrete pores are calculated explicitly from a set of formulated equations. The numerical analysis results are compared with two different sets of experimental data: (a) long-term (531 days) moisture migration test under a steady-state temperature of 200ºC, and (b) short-term (114 min) pressure build-up test under transient heating. These experiments were performed to investigate the moisture migration and pressure build-up in the concrete wall of a reactor containment vessel at high temperatures. The former experiment simulated the effect of long-term steady-state liner temperature during normal operation, and the latter simulated a situation where an accident resulted in sudden, short-term heating to approximately 400 ºC. Finally, concrete spalling is simulated by the numerical analysis; and the results show how the moisture content and pore pressure distributions in concrete exposed to fire change with time and temperature. The numerical analysis can predict the time, position and temperature at which spalling occurs.

Janotka [10] investigated on the effects of temperatures up to 800ºC on the strength characteristics, pore structure, and calculated permeability coefficients of concrete at the Mochovce nuclear power plant (Slovakia). No significant changes in modulus of elasticity, strength, average pore radius, or calculated permeability coefficients for tested specimens exposed to temperatures up to 400ºC were found. The effect of pore structure coarsening between 400 and 800ºC evidently results in a significant concrete strength decrease. The collapse of the concrete’s structural integrity characterizes the 800ºC temperature exposure.

Li and Purkiss [11] presented the critical review of the currently available models for the mechanical behaviour of concrete at elevated temperatures. Based on these models and experimental data a stress–strain–temperature model is proposed which incorporates the effect of transient strain implicitly. This model can be easily incorporated into existing commercial finite element analysis software. A numerical example on a wall element heated on two opposite faces indicates that at very early stages of heating transient strain does not play an important part, but that as the exposure time increases the effect of ignoring transient strain progressively increases and produces unconservative estimates of load carrying capacity. It would appear that at high temperatures the stress–strain curves in EN 1992-1-2 are unsafe if there are high axial loads due to high peak strains. It is observed that ignoring transient strain where there are significant axial compressive loads is unsafe.

Noumowe [12] showed the significant contribution of polypropylene fibre to the spalling resistance of high strength concrete. This investigation develops some important data on the mechanical properties and microstructure of high strength concrete incorporating polypropylene fibre exposed to elevated temperature up to 200ºC. When polypropylene fibre high strength concrete is heated up to 170ºC, fibres readily melt and volatilise, creating additional porosity and small channels in the concrete. DSC and TG analysis showed the temperature ranges of the decomposition reactions in the high strength concrete. SEM analysis showed supplementary pores and small channels created in the concrete due to fibre melting. Mechanical properties of concrete were studied at room temperature and after exposure at 200ºC. The addition of polypropylene fibres (1.8 kg/m3) may lead to small changes in residual compressive strength, modulus of elasticity and splitting tensile strength due to fibres melting during heating. The heat resistance of the mechanical properties appeared to decrease when polypropylene fibres were incorporated into concrete.

Noumowe [13] reported on the data on the behavior of high strength concrete at high temperatures is of concern in predicting the safety of buildings and construction in response to certain accidents or particular service conditions. Investigations were carried out on the behavior of the three concretes (high strength concrete with and without polypropylene fibers and lightweight aggregate concrete). The three groups of specimens were subjected to identical testing conditions. After a heating and cooling cycle at 200ºC, mechanical tests were carried out. The density of the coarse aggregates had a great influence on the temperature resistance properties. The temperature differentials through the concrete specimen thickness depended on the aggregate type and the results indicated that they could be as high as 92ºC (more than 11ºC/cm of concrete). Thermal gradients were greater in lightweight aggregate concrete than in normal weigth aggregate and high strength concrete. Polypropylene fibers did not modify the residual mechnical properties of tested high strength concrete. The adding of polypropylene fibers significantly modified neither the compressive behavior nor the tensile behavior. The substitution of lightweight aggregate for the normal weight aggregate in high strength concrete can provide benefits from reduced density, but the concrete compressive strength reduced. Results indicated that lightweight aggregate concrete endured the effect of elevated temperature better than normal aggregate concrete.

Noumowe et al. [14] showed some differences in the properties and the behaviour at high temperature of two concretes (ordinary and high strength) made with the same calcareous aggregates. Cylindrical specimens (ϕ 160 mm × 320 mm and ϕ 110 mm × 220 mm) of normal concrete were subjected to high temperatures up to 600ºC at heating rates of 0.1 and 1ºC min^{-1}. After heating and cooling, compressive strengths and weight losses were measured. Mercury porosimetry measurements were performed on samples drilled out from the heated and cooled specimens. High temperature exposure leads to a reduction of residual strength of both normal and high strength of both normal and high strength concretes. There is a minor change in the residual compressive strength up to about 200ºC and rapid drop after 350ºC. About 40% is left after 600ºC.

Phan and Carino [15] presented a compilation of fire test data, which shows distinct behavioral differences between high-strength concrete (HSC) and normal strength concrete (NSC) at elevated temperature. The differences are most pronounced in the temperature range of 20 °C to 400 °C. HSC has a higher strength loss than NSC in the temperature range between 25 °C to 400 °C. HSC is more susceptible to explosive spalling when exposed to high temperature (above 300 °C). Data for LWA HSC under all three-test conditions are scarce, as is data for NWA HSC under *stressed* test conditions. The modulus of elasticity of HSC and NSC vary similarly with temperature, but LWA HSC retains higher proportions of the original elastic modulus at high temperature than NWA HSC. Current fire design provisions of codes such as the CEN Eurocodes and the CEB are unconservative for estimating mechanical properties of HSC at elevated temperatures. The Finnish design curve is more applicable, but it is still slightly unconservative, especially in the temperature range of 200 °C to 400 °C. The basic understanding of how high temperatures cause explosive spalling in HSC has not been completely developed.

Phan et al. [16] reported on effects of elevated temperature exposure on residual mechanical properties of High Performance Concrete (HPC). Heating the 102 x 204 mm cylinders to steady state thermal conditions at a target temperature, and loading them to failure measured residual mechanical properties after the specimens had cooled to room temperature. The test specimens were made of four HPC mixtures with water-to-cementitious material ratio *(w/cm)* ranging from 0.22 to 0.57, and room-temperature compressive strength at testing ranges from 51 MPa to 93 MPa. Two of the four HPC mixtures contained silica fume. The specimens were heated to a maximum core temperature of 450ºC, at a heating rate of 5ºC/min. Experimental results indicate that HPCs with higher original strength (lower *w/cm)* and with silica fume retain more residual strength after elevated temperature exposure than those with lower original strength (higher *w/cm)* and without silica fume. The differences in modulus of elasticity are less significant. However, the potential for explosive spalling increased in HPC specimens with lower *w/cm* and silica fume. An examination of the specimens' heating characteristics indicate that the HPC mixtures which experienced explosive spalling had a more restrictive process of capillary pore and chemically bound water loss than those which did not experience spalling.

Poon et al. [17] evaluated on the strength and the durability performance of the normal and high strength pozzolanic concretes incorporating silica fume, fly ash, and blast furnace slag was compared at elevated temperatures up to 800ºC. The strength properties were determined using an unstressed residual compressive strength test, while durability was investigated rapid chloride diffusion test, mercury intrusion porosimetry (MIP), and crack pattern observations. It was found that pozzolanic concretes containing fly ash and blast furnace slag give the best performance particularly at temperatures below 600ºC as compared to the pure cement concretes. Explosive spalling occurred in the high-strength concretes (HSCs) containing silica fume. A distributed network of fine cracks was observed in all fly ash and blast furnace slag concretes, but no spalling or splitting occurred. The high-strength pozzolanic concretes showed a serve loss in the permeability-related durability than compressive strength loss. Thirty percent replacement of cement by fly ash in HSC and 40% replacement of cement by blast furnace slag in normal-strength (NSC) were found to be retaining maximum strength durability after high temperatures. High temperatures can be divided into distinct ranges in terms of effect on concrete strength. In the range of 20-200ºC, an increase in strength was observed in fly ash concretes. At 400ºC, most HSCs maintained their original strength, while an average loss of 20% strength was observed in NSCs. After 400ºC, both types of concrete lost their strength rapidly and the rate of strength loss was more in HSC.

Poon et al. [18] presented the effects of elevated temperatures on the compressive strength stress-strain relationship (stiffness) and energy absorption capacities (toughness) of concrete. High performance concretes (HPCs) were prepared in the series, with different cementitious material constitutions using plain ordinary Portland cement (PC), with and without metakaolin (MK) and silica fume (SF) separate replacements. Each series comprised a concrete mix, prepared without any fibers, and concrete mixes reinforced with either or both steel fibers and polypropylene (PP) fibers. The results showed that after exposure to 600 and 800ºC, the concrete mixes retained, respectively, 45% and 23% of their compressive strength, on average. The results also show that after the concrete was exposed to the elevated temperatures, the loss of stiffness was much quicker than the loss in compressive strength, but the loss of energy absorption capacity was relatively slower. A 20% replacement of the cement by MK resulted in a higer compressive strength but a lower specific toughness, as compared with the concrete prepared with 10% replacement of cement by SF. The MK concrete also showed quicker losses in the compressive strength, elastic modulus and energy absorption capacity after exposure to the elevated temperatures. Steel fibers approximately doubled the energy absorption capacity of the unheated concrete. They were effective in minimizing the degradation of compressive strength for the concrete after exposure to the elevated temperatures. The steel fiber reinforced concretes also showed the highest energy absorption capacity after the high temperature exposure, although they suffered a quick loss of this capacity.

Ravindrarajah et al. [19] investigated that the degradation of the strengths and stiffness of high-strength concrete in relation to the binder material type. The results showed that the binder material type has a significant influence on the performance of high-strength concrete particularly at temperatures below 800^{o}C. The influence of the binder material type is significantly decreased at temperature of 1000^{o}C. The strengths and stiffness of high-strength concrete are reduced with the increase in temperature without any threshold temperature level. The strengths are susceptible to the elevated temperatures compared to stiffness of concrete. High-strength concrete containing silica fume seems to be more sensitive to elevated temperature.

Sakr and Hakim [20] presented the effect of different durations (1, 2 and 3 h) of high temperatures (250, 500, 750 and 950ºC) on the physical, mechanical and radiation properties of heavy concrete. The effect of fire fitting systems on concrete properties was investigated. Results showed that ilmenite concrete had the highest density; modulus of elasticity and lowest absorption percent, and it had also higher values of compressive, tensile, bending and bonding strengths than gravel or baryte concrete. Ilmenite concrete showed the highest attenuation of transmitted gamma rays. Firing (heating) exposure time was inversely proportional to mechanical properties of all types of concrete. Ilmenite concrete was more resistant to elevated temperature. Foam or air proved to be better than water as a cooling system in concrete structure exposed to high temperature because water leads to a big damage in concrete properties.

Savva et al. [21] performed the investigation to show the influence of high temperatures on the mechanical properties and properties that affect the measurement by non-destructive methods (rebound hammer and pulse velocity) of concrete containing various levels (10% and 30%) of pozzolanic materials. Three types of Pozzolans, one natural pozzolan and two lignite fly ashes (one of low and the other of high lime content) were used for cement replacement. Two series of mixtures were prepared using limestone and siliceous aggregates. The W/b and the cementitius material content were maintained constant for all the mixtures. Concrete specimens were tested at 100, 300, 600 and 750 ºC for 2 h without any imposed load, and under the same heating regime. At the age of 3 years, tests of compressive strength, modulus of elasticity, rebound hummer and pulse velocity were come out. Concretes with pozzolanic materials show better strength results than the pure OPC concretes, up to 300ºC, while they seem to be more sensitive when exposed to heating above 300ºC. Up to 300ºC, only a small part of the initial strength is changed. The type of binder and aggregate does not affect the strength change significantly in this temperature range. Between 100 and 300ºC, the initial strength of almost all the mixtures increases. This increase is higher for siliceous concretes. At temperature above 300ºC, a decrease in strength is observed, which is higher for the PFA (siliceous aggregates) concretes and for ME (limestone aggregates). At 600ºC, the strength of mixtures is reduced to about the half; at 750ºC, the reduction is from 75% to 93%. Temperatures between 300 and 750 may be regarded as critical to the strength loss of concrete. At these temperatures, the greater the percentage of the replacement of OPC becomes, the greater the reduction in initial strength tends to be. A continuous drop in the modulus of elasticity is noticed at all the temperatures. This drop is higher for the limestone concretes.

Schindler [22] investigated that the development of high concrete temperatures could cause a number of effects that have been shown to be detrimental to long-term concrete performance. High concrete temperatures increase the rate of hydration, thermal stresses, the tendency for drying shrinkage cracking, permeability, and decrease long-term concrete strengths, and durability as a result of cracking. Data from the Texas Rigid Pavement database was analyzed to reveal that there are an increased number of failures as the air temperature at placement increases. It was further shown that this was the case for both major coarse aggregate types: limestone and siliceous river gravel. The result of this analysis emphasizes the importance of concrete temperature control during concrete pavement construction in hot weather conditions. CRC pavement data from the Texas Rigid Pavement (TRP) database was used to evaluate the effect of concrete temperatures on long-term CRC pavement performance. It was shown for both major aggregate types that there are an increased number of failures as the air temperature at placement increases. More than 36% of all failures occurred in the sections that were placed under conditions where the air temperature at placement exceeded 32.0°C (90°F). Around 26% of the failures occurred when the sections were constructed at air temperature between 26.5°C and 32°C. The result of this analysis emphasizes the importance of concrete temperature control during concrete pavement construction in hot weather conditions.

Shin et al. [23] carried out investigations to produce own experimental data of physical properties of domestic concrete used in Korean nuclear power plants (NPPs), and to study on the thermal behavior of concrete exposed to high temperature conditions. The chemical composition of Korean concrete is similar to that of US basaltic concrete. The thermal properties of the concrete, such as density, conductivity, diffusivity, and specific heat were also measured with a wide temperature range of 20-1100ºC. Most thermo-physical properties of concrete decrease with an increase in temperature except for the specific heat, and particularly the conductivity and the diffusivity are a 50% lower at 900ºC as compared with the values at room temperature. The specific heat increases until 500ºC, decreases from 700 to 900ºC, and then increases again when temperature is above 900ºC. They have performed CORCON analysis and MCCI experiments to simulate a transient thermal behavior of concrete exposed to high temperature conditions. The measured maximum downward heat flux to the concrete specimen was estimated to be about 2.1 MW m^{-2} and the maximum erosion rate of the concrete to be 175 cm h^{-1} with maximum erosion depth of about 2 cm. In the CORCON analysis, it is found that the concrete compositions have an important effect upon concrete erosion.

Terro [24] reported that the effect of replacement of fine and coarse aggregates with recycled glass on the fresh and hardened properties of Portland cement concrete at ambient and elevated temperatures is studied. Percentages of replacement of 0–100% of aggregates with fine waste glass (FWG), coarse waste glass (CWG), and fine and coarse waste glass (FCWG) were considered. Soda-lime glass used for bottles was washed and crushed to fine and coarse aggregate sizes for use in the concrete mixes. Samples were cured under 95% RH at room temperatures (20–22ºC), heated in the oven to the desired temperatures, allowed to cool to ambient temperatures, and then tested for their residual compressive strength. The compressive strength of the concrete samples made with waste glass was measured at temperatures up to 700ºC. The results of this study showed that the compressive strength of concrete made with RG decreases up to 20% of its original value with increasing temperatures up to 700ºC. In general, concretes made with 10% aggregates replacement with FWG, CWG and FCWG had better properties in the fresh and hardened states at ambient and high temperatures than those with larger replacement. Concretes made with FWG aggregates had higher compressive strengths than those made with CWG and FCWG at ambient and elevated temperatures

Wong et al. [25] presented the basic damage mechanisms of concrete under fire attacks, and then the experimental study of the residual compressive strength and durability properties of normal and high strength concretes made of materials available in Hong Kong after exposure to high temperatures up to 800°C. The effects of post-fire-curing on the strength and durability recovery of fire-damaged concrete were also investigated. High temperatures can be divided into distinct ranges in terms of effect on concrete strength. In the range of 20°-200°C, an increase in strength was observed in PFA and GGBS concretes. At 400°C, most HSC maintained their original strength while an average loss of 20% strength was observed in NSC. After 400°C, both types of concrete lost their strength rapidly and the rate of strength loss was more in HSC. In HSC, the PFA concretes showed the best performance at elevated temperatures followed by GGBS, OPC and CSF concretes. The mix containing 30% PFA replacement gave the maximum relative residual strength. In NSC, the GGBS concretes gave the best performance followed by PFA and OPC concretes. The 40% replacement level was found to be optimum. The mechanical strength of HSC decreased in a similar manner to that of NSC when subjected to high temperatures up to 800°C. However, HSC maintained a greater proportion of its relative residual compressive strength than the NSC. The HSC suffered a marginally smaller loss of mechanical strength but a greater worsening of the permeability related durability than the NSC. Among HSC, the PFA concretes suffered the least damage in impermeability followed by GGBS, OPC and CSF concretes. In NSC, the sequence was GGBS, PFA and OPC concretes. Severe deterioration and spalling was observed in most CSF concretes and some HS-OPC concretes. Most of the spalling occurred between 400°-600°C. No spalling was observed in PFA or GGBS concretes. The PFA and GGBS concretes were found to be able to retain their properties better at elevated temperatures and can be used in those places where there is a high risk of fire. The CFS concrete with more than 5% replacement should be avoided at such places due to the high risk of explosive spalling.

CHAPTER 3

EXPERIMENTAL PROGRAMME

3.1 Object of testing

The main objective of testing was to know the behavior of concrete with replacement of cement with high volume fly ash at elevated temperature up to 120˚C.

The main parameters studied were compressive strength, split tensile strength, modulus of elasticity. The materials used for casting concrete samples along with tested results are described.

3.2 Test Results of Materials Used In Present Work

3.2.1 Cement

IS mark 43 grade cement (Brand-ACC cement) was used for all concrete mixes. The cement used was fresh and without any lumps. Testing of cement was done as per IS: 8112-1989 [26]. The cement used was similar to Type I cement (ASTM C 150). The various tests results conducted on the cement are reported in table 3.1.

Table 3.1 Properties of cement

S. No.	Characteristics	Values obtained	Standard value
1.	Normal consistency	34%	-
2.	Initial setting time (minutes)	48 min.	Not less than 30
3.	Final setting time (minutes)	240 min.	Not greater than 600
4.	Fineness (%)	3.5 %	<10
5.	Specific gravity	3.07	-

3.2.2 Coarse aggregates

Locally available coarse aggregates having the maximum size of 10 mm and 20mm were used in the present work. The 10mm aggregates used were first sieved through 10mm sieve and then through 4.75 mm sieve and 20mm aggregates were firstly sieved through 20mm sieve. They were then washed to remove dust and dirt and were dried to surface dry condition. The aggregates were tested per Indian Standard Specifications IS: 383-1970 [27]. The results of various tests conducted on coarse aggregate are given in Table 3.2, Table 3.3 and Table 3.4.

Table 3.2 Properties of Coarse aggregates

S. No.	Characteristics	Value
1.	Type	Crushed
2.	Maximum size	20 mm
3.	Specific gravity (10 mm)	2.704
4.	Specific gravity (20 mm)	2.825
5.	Total water absorption (10 mm)	1.6432 %
6.	Total water absorption (20 mm)	3.645 %
7.	Moisture content (10 mm)	0.806 %

8.	Moisture content (20 mm)	0.7049 %
9.	Fineness modulus (10 mm)	6.46
10.	Fineness modulus (20 mm)	7.68

Table 3.3 Sieve analysis of 10 mm aggregates

S. No.	Sieve No.	Mass Retained (kg)	percentage Retained, %	percentage Passing, %	Cumulative %age Retained
1.	80 mm	-	0.00	100	0.00
2.	40 mm	-	0.00	100	0.00
3.	20 mm	-	0 .00	100	0.00
	12.5 mm	0.555	18.5	81.5	18.5
4.	10 mm	0.8905	29.68	51.82	48.18
5.	4.75 mm	0.9565	31.88	19.94	80.06
11.	Pan	0.5970	19.90	0.04	99.96
				ΣC =	146.74

Fineness Modulus of Coarse aggregate(10 mm) = ΣC+500/100 = 146.74+500/ 100 = 6.46

Table 3.4 Sieve analysis of 20 mm aggregates

S. No.	Sieve No.	Mass Retained (kg)	percentage Retained, %	percentage Passing, %	Cumulative %age Retained
1.	80 mm	-	0.00	100	0.00
2.	40 mm	-	0.00	100	0.00
3.	20 mm	0	0 .00	100	0.00
4.	12.5 mm	2.1865	72.883	27.117	72.883
5.	10 mm	0.6745	22.483	4.634	95.366
6.	4.47 mm	0.1390	4.633	0.01	99.999
11.	Pan	0	0.00	-	-
				ΣC =	268.244

Fineness Modulus of Coarse aggregate(20mm)= ΣC+500/100 = 268.244+500/ 100 = 7.68

3.2.3 Fine Aggregate

The sand used for the experimental programme was locally procured and conformed to grading zone III.

The sand was first sieved through 4.75 mm sieve to remove any particles greater than 4.75 mm and then was washed to remove the dust. The fine aggregates were tested per Indian Standard Specifications IS: 383-1970 [27]. Properties of the fine aggregate used in the experimental work are tabulated in Table 3.5 and Table 3.6.

Table 3.5 Properties of fine aggregates

S. No.	Characteristics	Value
1.	Type	Uncrushed (natural)
2.	Specific gravity	2.68
3.	Total water absorption	1.02 %
4.	Moisture content	0.16 %
5.	Net water absorption	0.86 %
6.	Fineness modulus	2.507
7.	Grading zone	III

Table 3.6 Sieve analysis of fine aggregate

S. No.	Sieve No.	Mass Retained (gms)	percentage Retained,%	percentage Passing, %	Cumulative %age Retained
1.	4.75 mm	95.0	9.5	90.5	9.5
2.	2.36 mm	42.5	4.25	86.25	13.75
3.	1.18 mm	110.5	11.05	75.2	24.8
4.	600 µm	128.5	12.85	62.35	37.65
5.	300 µm	308.0	30.8	31.55	68.45
6.	150 µm	281.0	28.1	3.45	96.55
7.	Pan	34.5	3.45	-	
				ΣF =	250.7

Fineness Modulus of fine aggregate = ΣF/100 = 250.7/100 = 2.507

3.2.4 Fly ash

Investigations were made on fly ash procured from Guru Gobind Singh Super Thermal Power Plant, Ropar, and Punjab. It was tested for chemical and physical properties per ASTM C 311.The chemical and physical properties of the fly ash used in this investigation are listed in Table 3.7 and Table 3.8 respectively.

Table 3.7 Chemical Composition of Fly Ash

S.N.	***Particulars***	**Requirement ASTM C 618 (%)**	**Test Results (%)**
1.	(SiO_2+Al_2O_3+Fe_2O_3), %	70.0 min	91.69
2.	SiO_2, %	35.0 min	59.08
3.	MgO	5.0 max	0.36
4.	Sulphuric Anhydride, %	3.0 max	0.11
5.	Total Alkali as Na_2O, %	1.5 max	0.62
6.	Total Loss on Ignition, %	5.00 max	2.08

Table 3.8 Physical Properties of Fly Ash

S.N.	***Particulars***	**Requirement ASTM C 618**	**Test Results**
1.	Fineness Specific Surface (cm^2/gm)	3200 min	3258
2.	Residue on 45 micron (wet sieving)	34 max	30.17
3.	Lime Reactivity (kg/cm^2)	45 min	51.03
4.	Compressive strength (kg/cm^2), 28 days	Not less than 80% of strength of corresponding plain cement Mortar cubes	85.99
5.	Dry shrinkage, %	0.15 max	0.04
6.	Soundness expansion by auto clave, %	0.8 max	0.03

3.2.5 Water

Potable tap water was used for the concrete preparation and for the curing of specimens.

3.2.6 Superplasticizer

Conplast- SP430, a concrete superplasticizer based on Sulphonated Naphthalene Polymer was used as a water-reducing admixture and to improve the workability of fly ash concrete. Conplast SP430 has been specially formulated to give high water reductions up to 25% without loss of workability or to produce high quality concrete of reduced permeability. Conplast SP430 is non-toxic.

Superplasticizer complies with IS: 9103:1999,ASTM C-494 Type F, BS 5057 part III [28]. The dosage of superplasticizer varied between 0.5% to 2% by weight of cement in plain cement concrete, high volume fly ash concrete. Technical data of Superplasticizer are listed in Table 3.9.

Table 3.9 Technical data of Superplasticizer

S. No.	Characteristics	Value
1.	Colour	Dark Brown liquid
2.	Specific gravity @ 30˚ C	1.22 to 1.225
3.	Air entrainment	Maximum 1 %
4.	Chloride content	Nil to BS 5075: 1982

3.3 Mix design

Concrete mix has been designed based on Indian Standard Recommended Guidelines .The proportions for the concrete, as determined were 1:1.45:2.2:1.103 with a water cement ratio of 0.5 by weight. One control mixture M-0 was designed per Indian Standard Specifications IS: 10262-1982 [29] to have 28-day compressive strength of

23.05 MPa. The other concrete mixtures were made by replacing cement with 30%, 40%, and 50% of Class F fly ash by mass. In doing so, water-to-cementitious materials ratio was kept almost same to investigate the effects of replacing cement with high volumes of Class F fly ash when other parameters were almost kept same. The mix designation and quantities of various materials for each designed concrete mix have been tabulated in Table 3.9 and 3.10.

Table 3.9 Mix Designation

Grade of concrete	**Concrete Type**	**Temperature,ºC**	**Designation**	**Percentage binder ratio**	
				Cement	**Fly ash**
M-20	Control Mix	N*	M-0	100	0
	Fly ash concrete	80	M-1	100	0
		100	M-2	100	0
		120	M-3	100	0
		N	M-4	70	30
		80	M-5	70	30
		100	M-6	70	30
		120	M-7	70	30

		N	M-8	60	40
		80	M-9	60	40
		100	M-10	60	40
		120	M-11	60	40
		N	M-12	50	50
		80	M-13	50	50
		100	M-14	50	50
		120	M-15	50	50

where, N* is the room temperature

Table 3.10 Proportion of M-20 Grade Concrete

	Cement kg/m^3	Fine Agg. kg/m^3	Course Agg. (10mm) kg/m^3	Course Agg. (20mm) kg/m^3	Fly Ash kg/m^3	Water (lts/m^3)	Plasticizer (lts/m^3)	Temp., ^oC	Slump, mm
M-0	372	538.45	410.4	818.85	0	186	4.464	N	45
M-1	372	538.45	410.4	818.85	0	186	4.464	80	45
M-2	372	538.45	410.4	818.85	0	186	4.464	100	45
M-3	372	538.45	410.4	818.85	0	186	4.464	120	45
M-4	260.4	538.45	410.4	818.85	111.6	186	5.022	N	45
M-5	260.4	538.45	410.4	818.85	111.6	186	5.022	80	45
M-6	260.4	538.45	410.4	818.85	111.6	186	5.022	100	45
M-7	260.4	538.45	410.4	818.85	111.6	186	5.022	120	45
M-8	223.2	538.45	410.4	818.85	148.8	186	5.394	N	45
M-9	223.2	538.45	410.4	818.85	148.8	186	5.394	80	45
M-10	223.2	538.45	410.4	818.85	148.8	186	5.394	100	45
M-11	223.2	538.45	410.4	818.85	148.8	186	5.394	120	45
M-12	186	538.45	410.4	818.85	186	186	6.882	N	45
M-13	186	538.45	410.4	818.85	186	186	6.882	80	45
M-14	186	538.45	410.4	818.85	186	186	6.882	100	45
M-15	186	538.45	410.4	818.85	186	186	6.882	120	45

3.4 Preparation Testing of specimens

Cylindrical mould of size 150 mm×300 mm were used to prepare the concrete specimens for the determinations of compressive strength, split tensile strength and modulus of elasticity of fly ash concrete. All specimens were prepared in accordance with Indian Standard Specifications IS: 516-1959 [30]. All the moulds were cleaned and oiled properly. These were securely tightened to correct dimensions before casting. Care was taken that there is no gaps left from where there is any possibility of leakage out of slurry. Concrete cylinders, 150 mm×300 mm were tested for the determinations of compressive strength, split tensile strength and modulus of elasticity of fly ash concrete as per Indian Standard Specifications IS: 516-1959 [30].

3.5 Batching, Mixing and Casting of Specimens

A careful procedure was adopted in the batching, mixing and casting operations. The coarse aggregates and fine aggregates were weighed first with an accuracy of 0.5 grams. The concrete mixture was prepared by hand mixing on a watertight platform. The fly ash and cement were mixed dry to uniform colour separately. Superplasticizer as per requirement was added to required quantity of water separately in different containers. On the watertight platform, the coarse and fine aggregates were mixed thoroughly. To this mixture, the mixture of cement and fly ash was added. These were mixed to uniform colour. Then water was added carefully so that no water was lost during mixing. Nine clean and oiled moulds for each category were then placed on the vibrating table respectively and filled in three layers. Vibrations were stopped as soon as the cement slurry appeared on the top surface of the mould.

The specimens were allowed to remain in the steel mould for the first 24 hours at ambient condition. After that these were demoulded with care so that no edges were broken and were placed in the curing tank at the ambient temperature for curing. The ambient temperature for curing was 27 ± 2^{0} C.

3.6 Heating and cooling regimes

At the age of 28 and 56 days, specimens were heated in an electric oven up to 80ºC, 100ºC, and 120ºC. Each temperature was maintained for 1 hour to achieve the thermal steady state. The heating rate was set at 2.5ºC/min. the specimens were allowed to cool naturally to room temperature.

3.7 Fresh concrete properties

Fresh concrete properties, such as slump, unit weight, temperature, and air content, were determined per Indian Standard Specifications IS: 1199-1959 [31]. The results are presented in Table 3.10.

CHAPTER – 4

RESULTS AND DISCUSSION

4.1 General

Temperature is one of the main factors that influence the strength. High temperature induces a loss of strength (both in compression and tension) and stiffness (Young's modulus). At high temperatures, chemical transformation of the gel weakened the matrix bonding, which brought about a loss of strength of fly ash concrete.

4.2 Compressive Strength

In this research, the values of compressive strength for different fly ash contents (0%, 30%, 40% and 50%) incorporating different temperature (40ºC, 80ºC ,100ºC, and 120ºC) at the end of different curing periods (28 days, 56 days) are given in Table 4.1.The results have also been plotted in Figs. 4.1 to 4.12, which shows the variation of compressive strength with cement replacements at different curing ages respectively and variation of compressive strength for different fly ash percent incorporating different degree of temperature.

Fig. 4.1 to 4.4 shows the variation of compressive strength with replacements with Class F fly ash at various temperatures (40ºC, 80ºC, 100ºC, and 120ºC). The compressive strength was calculated as the average of three cylinder tests. It is evident from Fig. 4.1 that compressive strength of concrete mixtures with 30%, 40% and 50 % of fly ash as cement replacement was lower than the control mixture (M-0) at all ages and that the strength of all mixtures continued to increase with the age. With the increase in temperature, compressive strength of concrete mixes with 30%, 40% and 50 % of fly ash as cement replacement decreased.

Fig. 4.1 shows that compressive strength decreased with the increase fly ash at different temperature. Compressive strength also decreased with the increase in temperature. At 120ºC temperature, the compressive strength decreased by 11.4%, 30.1%, 28.9%, and 27.5% when compared to normal temperature for 0%, 30%, 40%, and 50% replacement of fly ash with cement respectively at 56 days. At high temperatures, chemical transformation of gel weakened the matrix bonding, which brought about a loss of strength of fly as concrete.

Figs. 4.5 to 4.8 shows the compressive strength ratio (at 28 and 56 days) with respect to percentage replacement of cement by fly ash. Compressive strength at 56 days was 21.7 %, 35.6 %, 24.6% and 19.1 % higher than the 28 days compressive strength at normal temperature whereas it increased by 22.1%, 35.1%, 20.1%, and 13.9% at 80ºC. Similarly compressive strength increased at 100ºC and 120ºC with age.

Figs. 4.9 to 4.12 shows the compressive strength ratio (at 28 and 56 days) with respect to different temperature. Compressive strength increased with age at different temperature.

All the tests performed so far and reported here showed a brittle-type failure after cycles at 80˚C, 100˚C, and 120˚C, with very steep descending branches, more or less pronounced snapbacks and little differences compared to the tests at room temperature. The specimens failed after the formation of a number of longitudinal (vertical) cracks in the loading direction, and no shear type failures occurred.

In the tests performed at the higher temperatures, the material softened and the descending branches had a definitely negative slope, with ductile failures. However, the failure was always preceded by the formation of longitudinal cracks, with some differences compared to lower temperature tests.

4.3 Split tensile strength

It was found that split tensile strength of Class F fly ash concrete (using 30 %, 40 % and 50 % fly ash and a w/c of 0.5) at different temperature depended on the percentage of fly ash used and temperature. The variation of split tensile strength was shown in table 4.2.the variation in splitting tensile strength with fly ash content and temperature was similar to that observed in case of compressive strength.

Fig. 4.13 to 4.16 shows the variation of split tensile strength with replacements with Class F fly ash at various temperatures (40ºC, 80ºC, 100ºC, and 120ºC). Fig. 4.13 shows that split tensile strength decreased with the increase fly ash at different temperature. It also decreased with the increase in temperature. For control mix, split tensile strength was decrease by 28.6%, 32.1%, and 42.8% with respect to normal temperature at 80ºC, 100ºC, and 120ºC respectively at 28 days.

However, split tensile strength was found to increase with age as it was shown in Figs. 4.17 to 4.20 with respect to percentage replacement of cement by fly ash. At 56 days, mixtures M-3 (0% fly ash at 120˚C), M-7 (30% fly ash at 120˚C), M-11 (40% fly ash at 120˚C), and M-15 (40% fly ash at 120˚C) achieved split tensile strength of 2.4 MPa, 2.0MPa, 1.3 MPa, and 0.7 MPa, respectively; an increase of 33.3%, 40%, 46.1%, and 57.1%, respectively, for mixtures M-3 (0% fly ash at 120˚C), M-7 (30% fly ash at 120˚C), M-11 (40% fly ash at 120˚C), and M-15 (40% fly ash at 120˚C) in comparison with 28 days strength. Figs. 4.21 to 4.24 shows the split tensile strength ratio (at 28 and 56 days) with respect to different temperature. It increased with age at different temperature.

4.4 Modulus of elasticity

In this investigation, the modulus of elasticity, which is also called secant modulus, is taken as the slope of the chord from the origin to some arbitrary point on the stress-strain curve. The secant modulus calculated in this study is for 33% of the maximum stress. Modulus of elasticity of concrete mixtures was determined at the ages of 28 and 56 days. Results are given in the Table 4.3 and shown in Figs 4.25 to 4.36.

Fig. 4.25 to 4.28 shows the variation of modulus of elasticity with replacements with Class F fly ash at various temperatures (40ºC, 80ºC, 100ºC, and 120ºC). Test results indicated that the use of large proportion of fly ash reduced the modulus of concrete compared to that of control mixture. It also decreased with the increase in temperature. For control mix, modulus of elasticity was decrease by 18.7%, 32.1%, and 37.4% with respect to normal temperature at 80ºC, 100ºC, and 120ºC respectively at 56 days.

Figs. 4.29 to 4.32 shows the modulus of elasticity ratio (at 28 and 56 days) with respect to percentage replacement of cement by fly ash. At 56 days, mixtures M-3 (0% fly ash at 120˚C), M-7 (30% fly ash at 120˚C), M-11 (40% fly ash at 120˚C), and M-15 (40% fly ash at 120˚C) achieved modulus of elasticity of 16.4 GPa, 15.8 GPa, 15.2 GPa, and 7.6 GPa, respectively. The results indicated that modulus of elasticity of concrete mixtures increased with the age with respect to the replacement of fly ash. Figs. 4.33 to 4.36 shows the modulus of elasticity ratio (at 28 and 56 days) with respect to different temperature. Modulus of elasticity increased with age at different temperature.

Table 4.1: Compressive Strength (MPa) of Fly Ash Concrete

Fly Ash Content, %	Temperature,ºC	Designation	Compressive Strength,MPa	
			28 days	56 days
0	N	M-0	23.0	29.4
0	80	M-1	22.9	27.0
0	100	M-2	19.3	26.5
0	120	M-3	17.6	26.0
30	N	M-4	16.2	25.2
30	80	M-5	13.3	20.5
30	100	M-6	12.1	19.7
30	120	M-7	10.8	17.6
40	N	M-8	15.4	20.4
40	80	M-9	14.3	17.9
40	100	M-10	12.9	17.8
40	120	M-11	12.8	14.5
50	N	M-12	10.8	13.3
50	80	M-13	10.5	12.2
50	100	M-14	8.9	11.3
50	120	M-15	8.4	9.6

Table 4.2: Split tensile strength (MPa) of Fly Ash Concrete

Fly Ash Content, %	Temperature,ºC	Designation	Split tensile strength,MPa

			28 days	56 days
0	N	M-0	2.8	3.0
0	80	M-1	2.0	2.8
0	100	M-2	1.9	2.5
0	120	M-3	1.6	2.4
30	N	M-4	1.7	2.7
30	80	M-5	1.4	2.3
30	100	M-6	1.3	2.2
30	120	M-7	1.2	2.0
40	N	M-8	1.4	2.1
40	80	M-9	1.1	1.6
40	100	M-10	1.0	1.5
40	120	M-11	0.7	1.3
50	N	M-12	0.7	1.4
50	80	M-13	0.6	1.1
50	100	M-14	0.5	0.9
50	120	M-15	0.3	0.7

Table 4.3:Modulus of elasticity (GPa) of Fly Ash Concrete

Fly Ash Content, %	**Temperature,ºC**	**Designation**	**Modulus of elasticity,GPa**	
			28 days	**56 days**
0	N	M-0	17.6	26.2
0	80	M-1	15.1	21.3
0	100	M-2	14.7	17.8
0	120	M-3	11.6	16.4
30	N	M-4	10.9	19.4
30	80	M-5	9.6	16.3
30	100	M-6	9.5	16.2
30	120	M-7	9.9	15.8
40	N	M-8	7.7	15.8
40	80	M-9	6.9	15.4
40	100	M-10	6.1	15.3
40	120	M-11	5.1	15.2
50	N	M-12	5.9	8.5
50	80	M-13	5.7	7.9
50	100	M-14	4.8	7.7
50	120	M-15	4.4	7.6

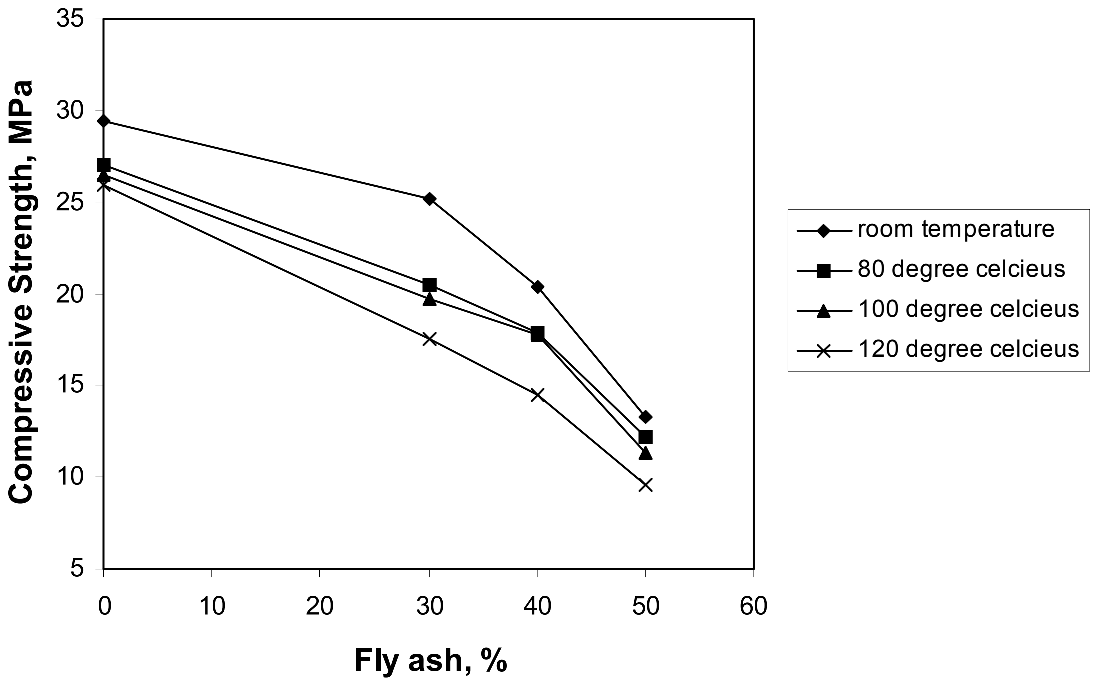

Fig. 4.1 Compressive Strength vs Fly ash (56 days)

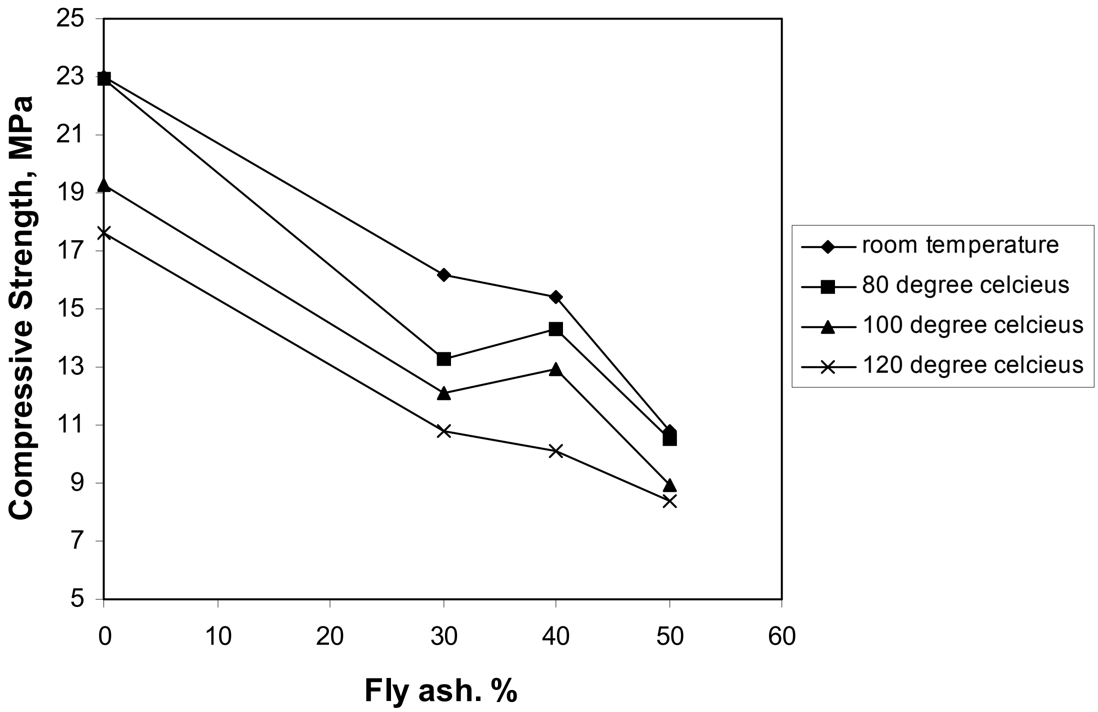

Fig. 4.2 Compressive Strength vs Fly ash (28 days)

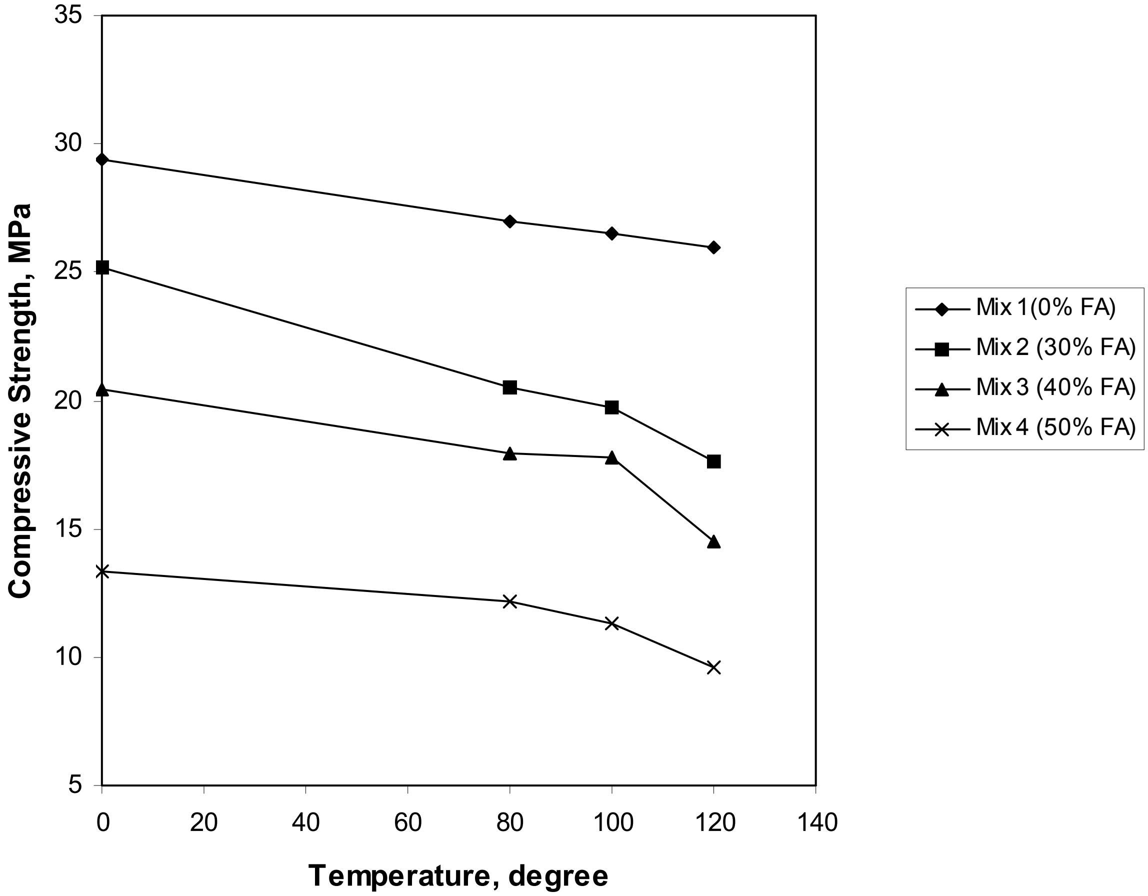

Fig. 4.3 Compressive Strength vs Temperature (56 days)

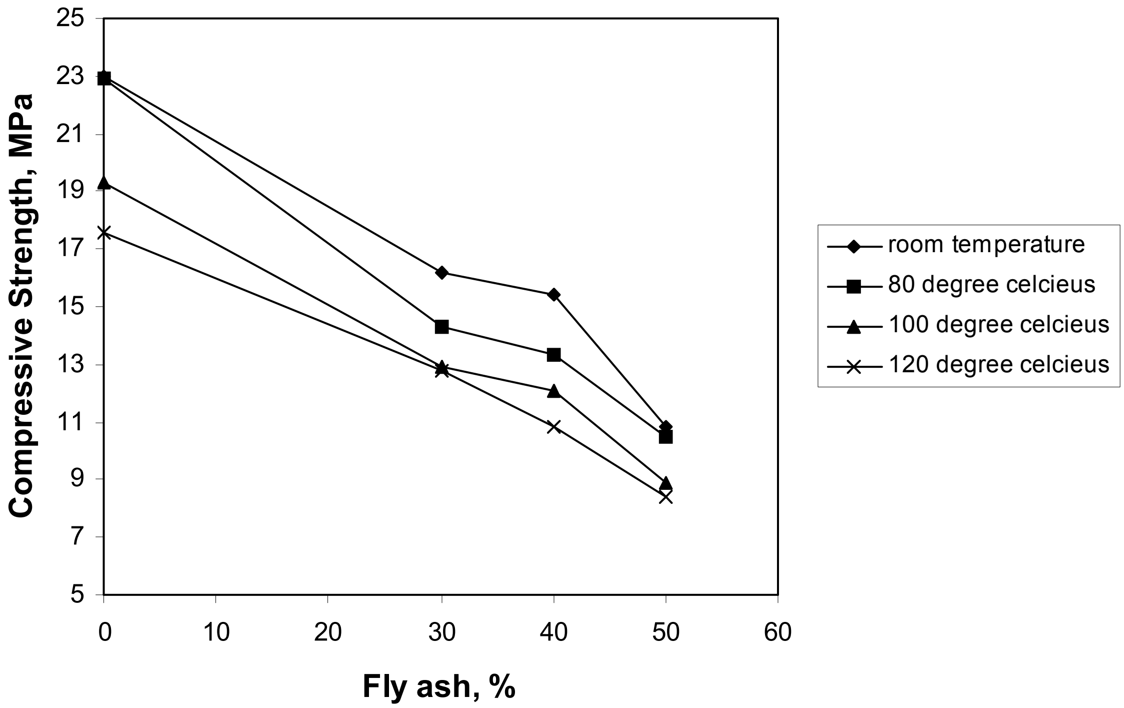

Fig. 4.4 Compressive Strength vs Temperature (28 days)

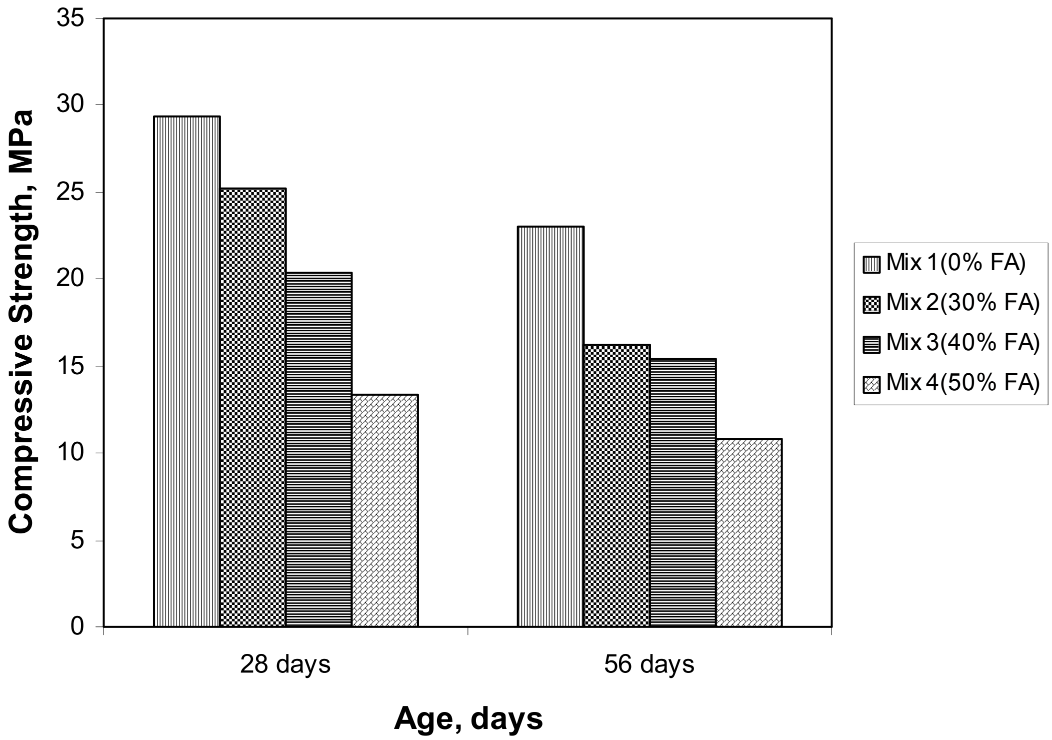

Fig. 4.5 Compressive Strength vs Age (at room temperature)

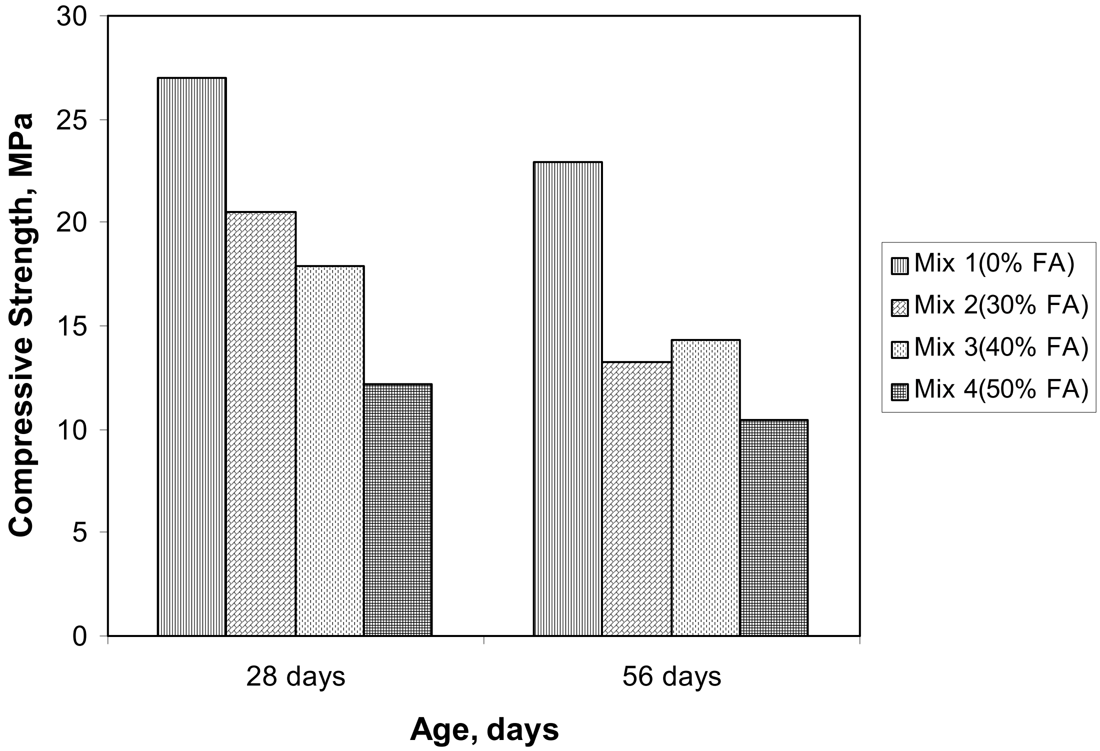

Fig. 4.6 Compressive Strength vs Age (at 80˚C)

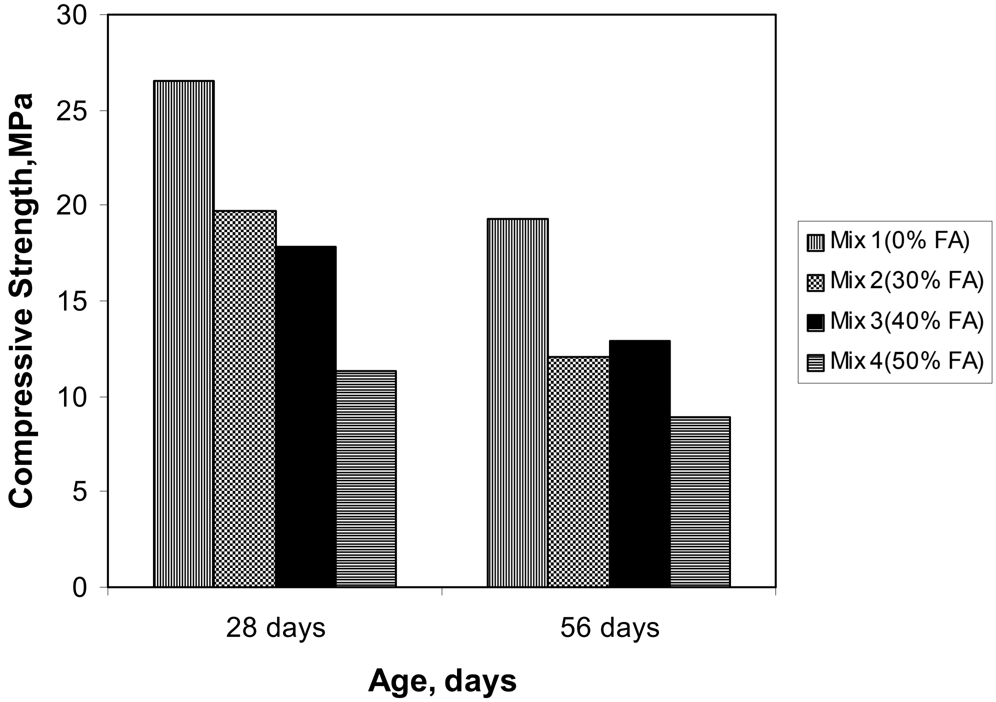

Fig. 4.7 Compressive Strength vs Age (at 100˚C)

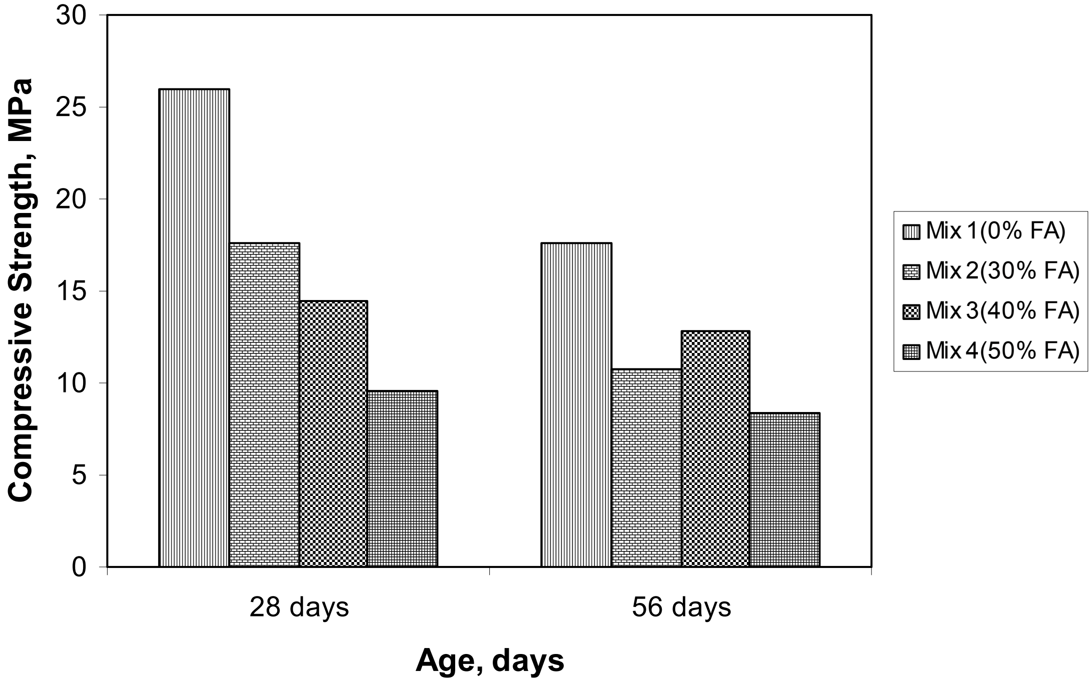

Fig. 4.8 Compressive Strength vs Age (at 120˚C)

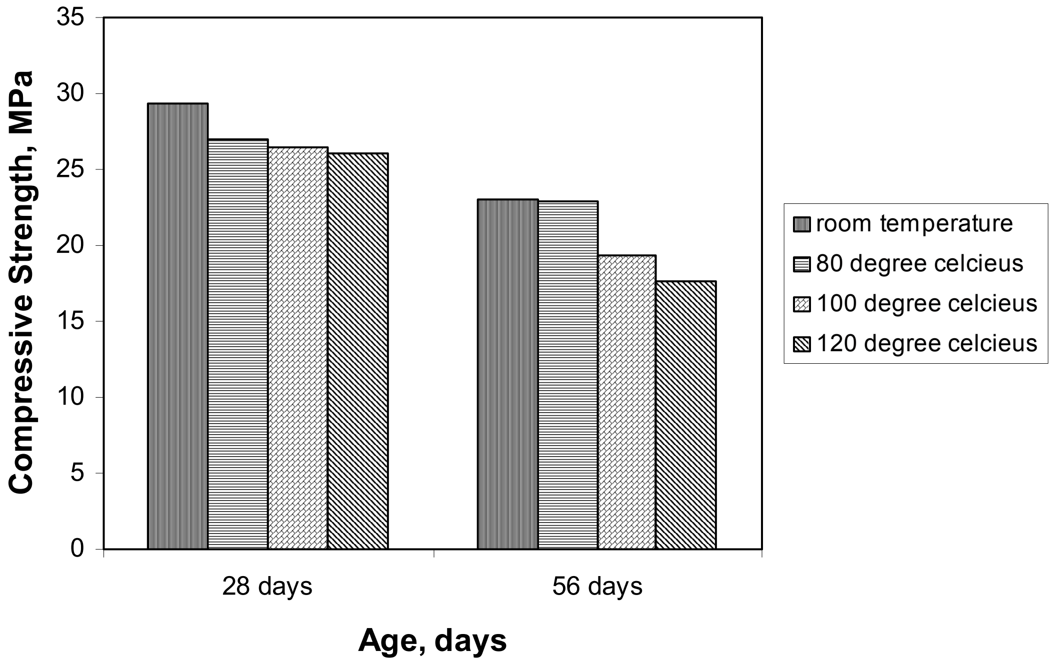

Fig. 4.9 Compressive Strength vs Age (without Fly ash)

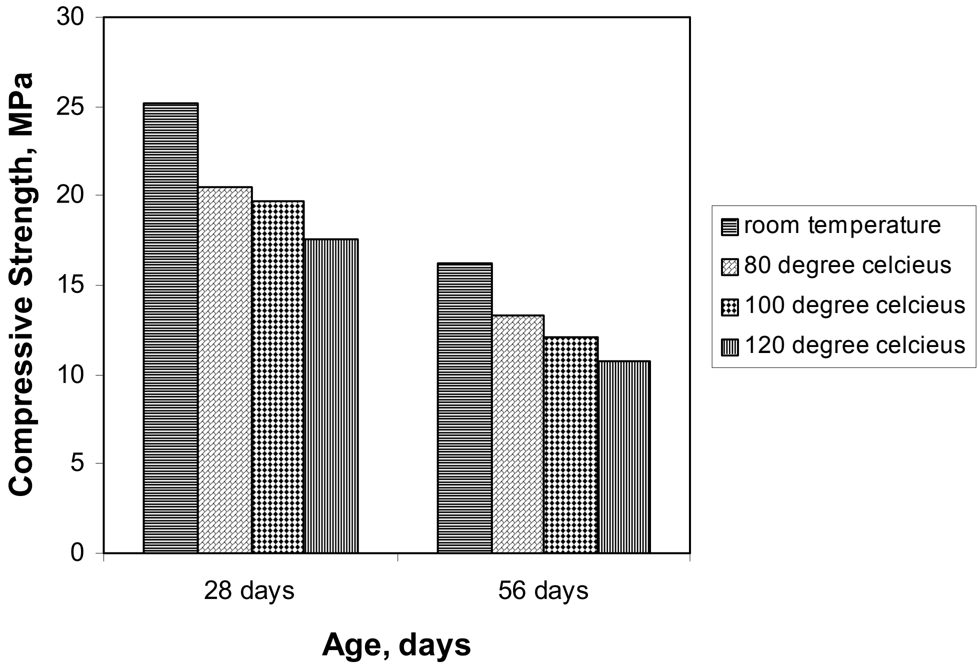

Fig. 4.10 Compressive Strength vs Age (with 30% Fly ash)

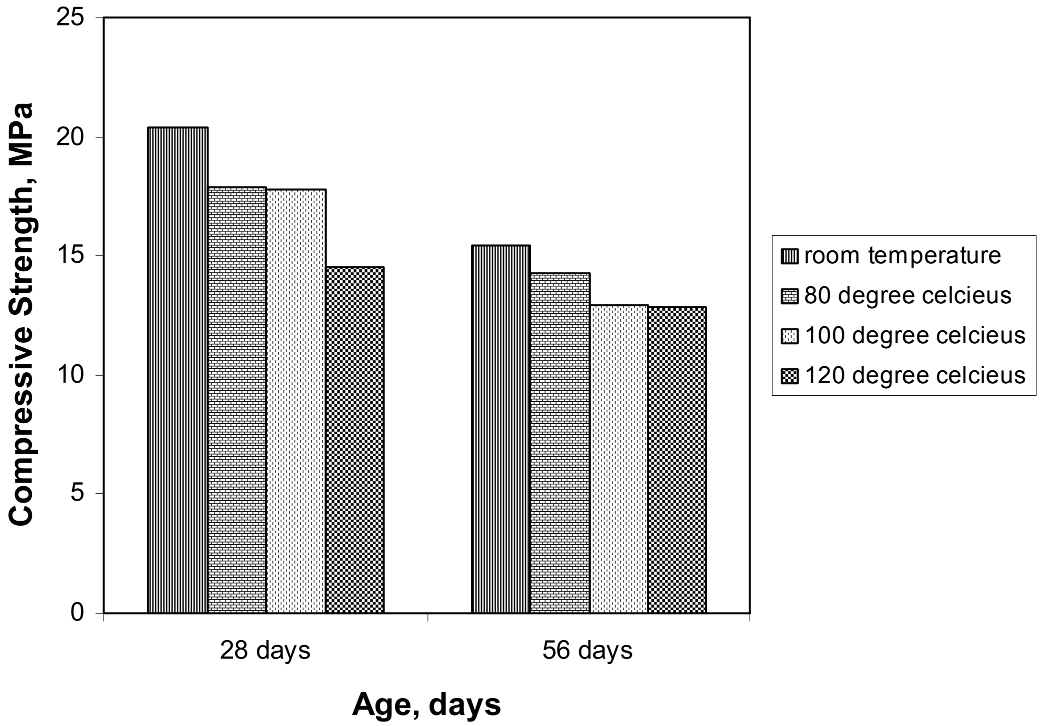

Fig. 4.11 Compressive Strength vs Age (with 40% Fly ash)

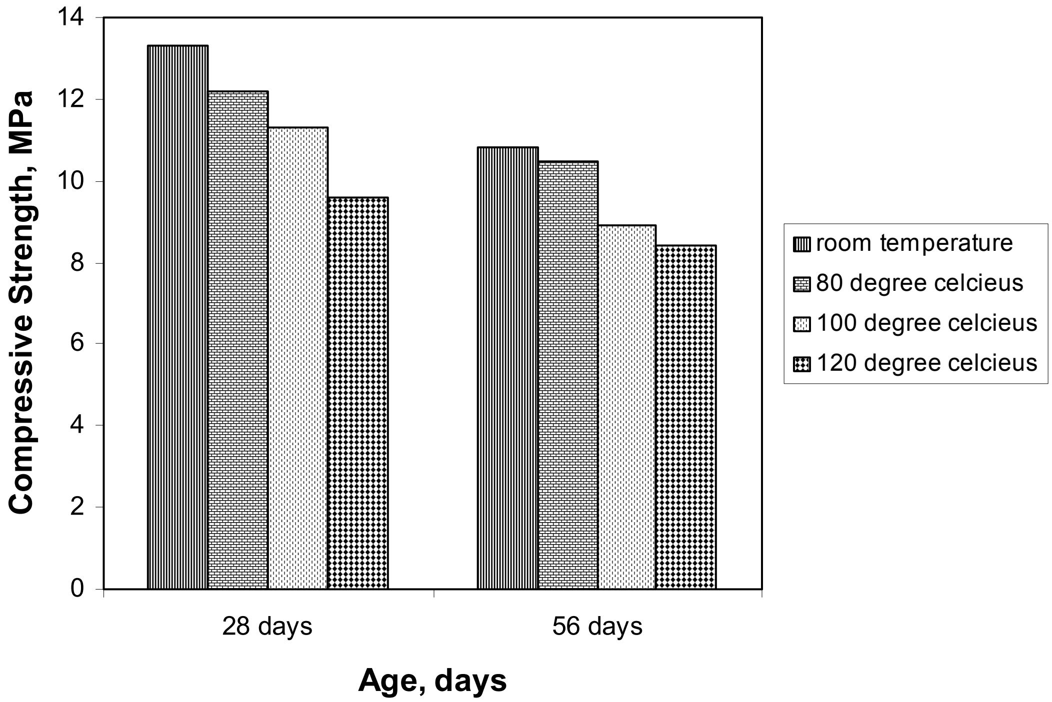

Fig. 4.12 Compressive Strength vs Age (with 50% Fly ash)

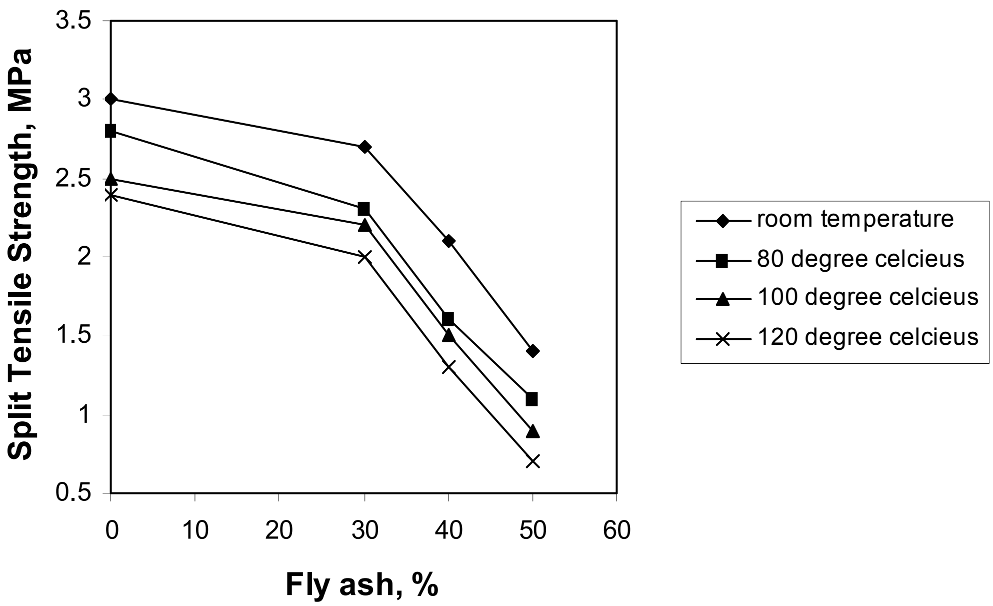

Fig. 4.13 Split Tensile Strength vs Fly ash (56 days)

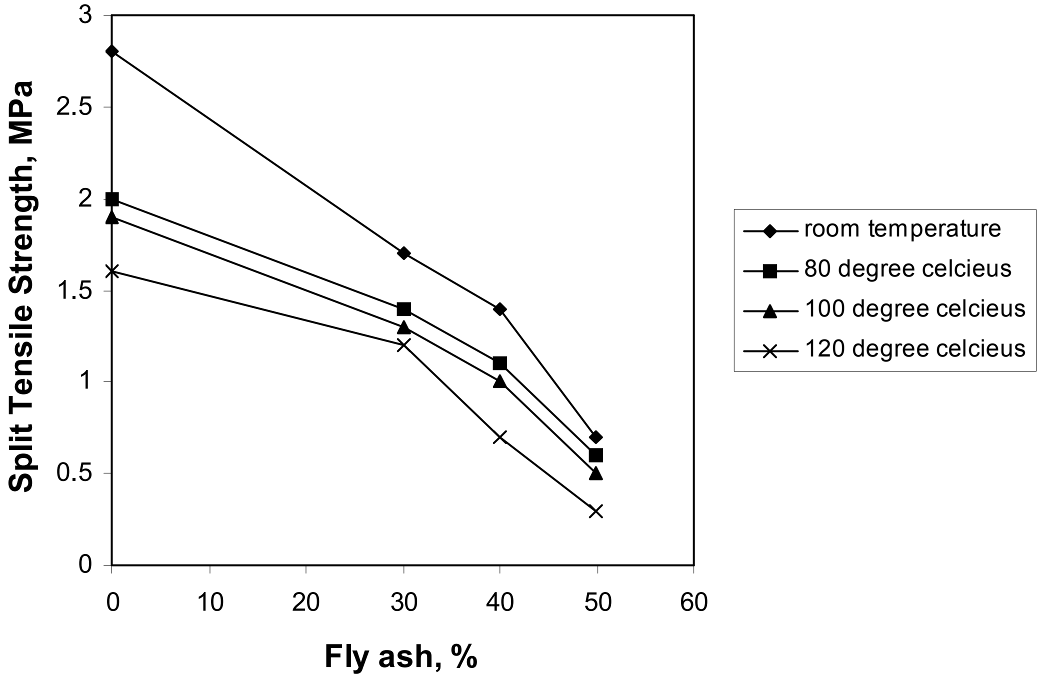

Fig. 4.14 Split Tensile Strength vs Fly ash (28 days)

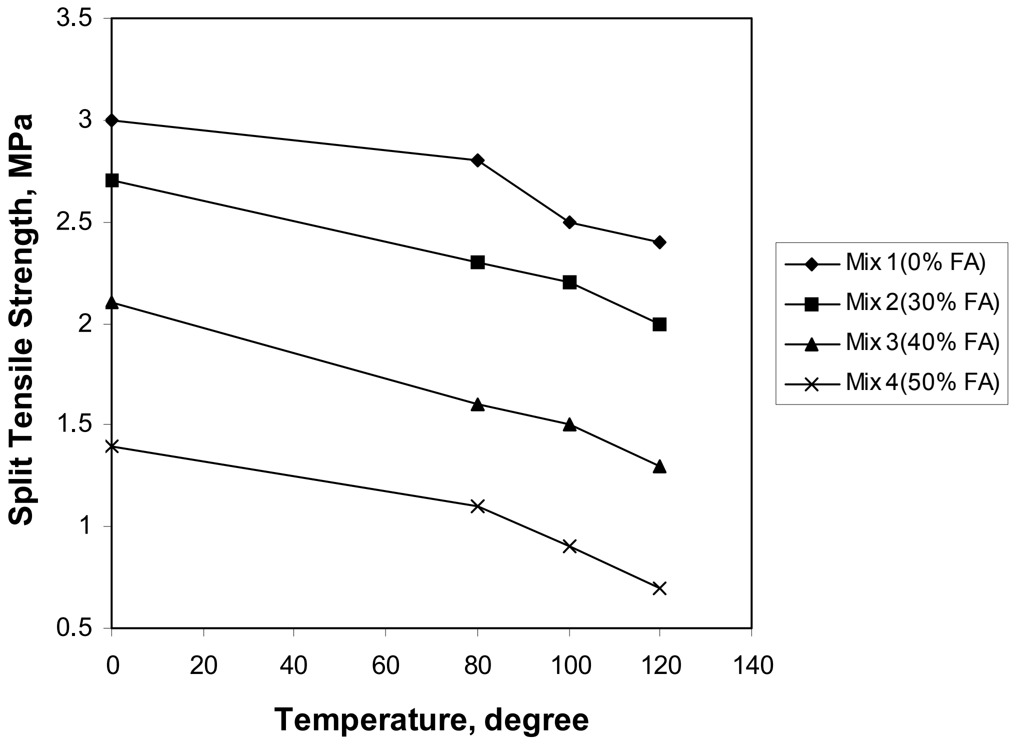

Fig. 4.15 Split Tensile Strength vs Temperature (56 days)

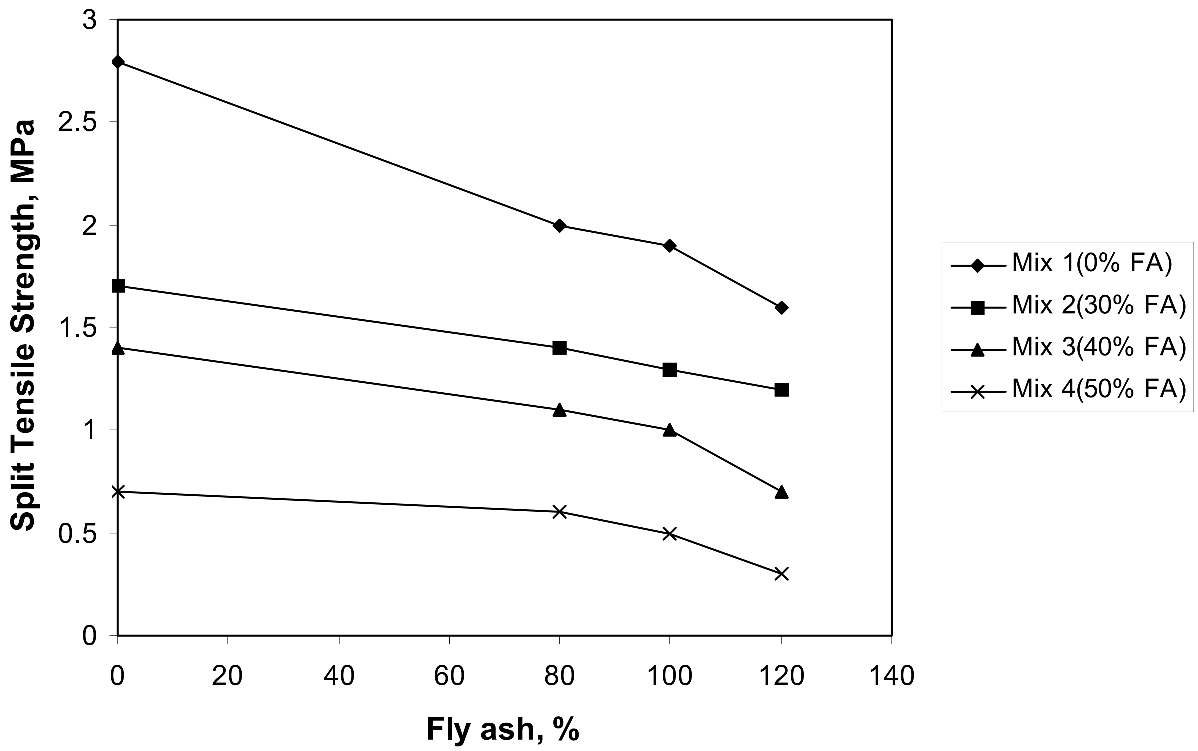

Fig. 4.16 Split Tensile Strength vs Temperature (28 days)

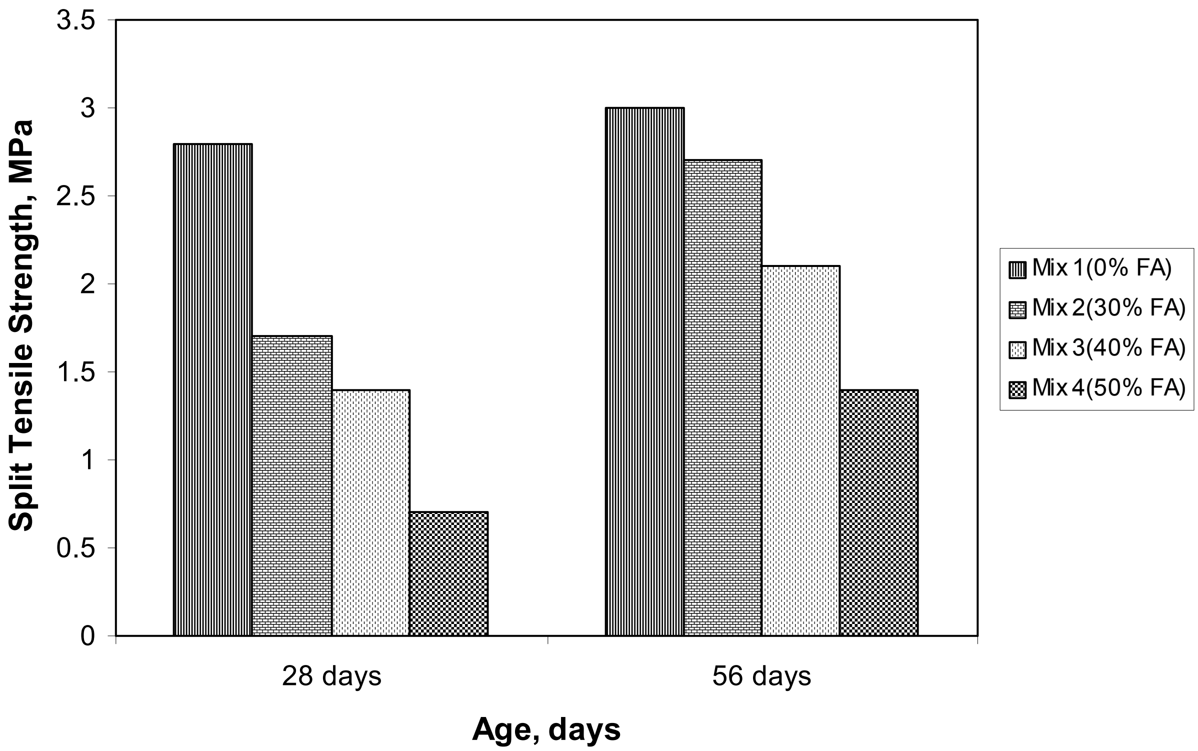

Fig. 4.17 Split Tensile Strength vs Age (at room temperature)

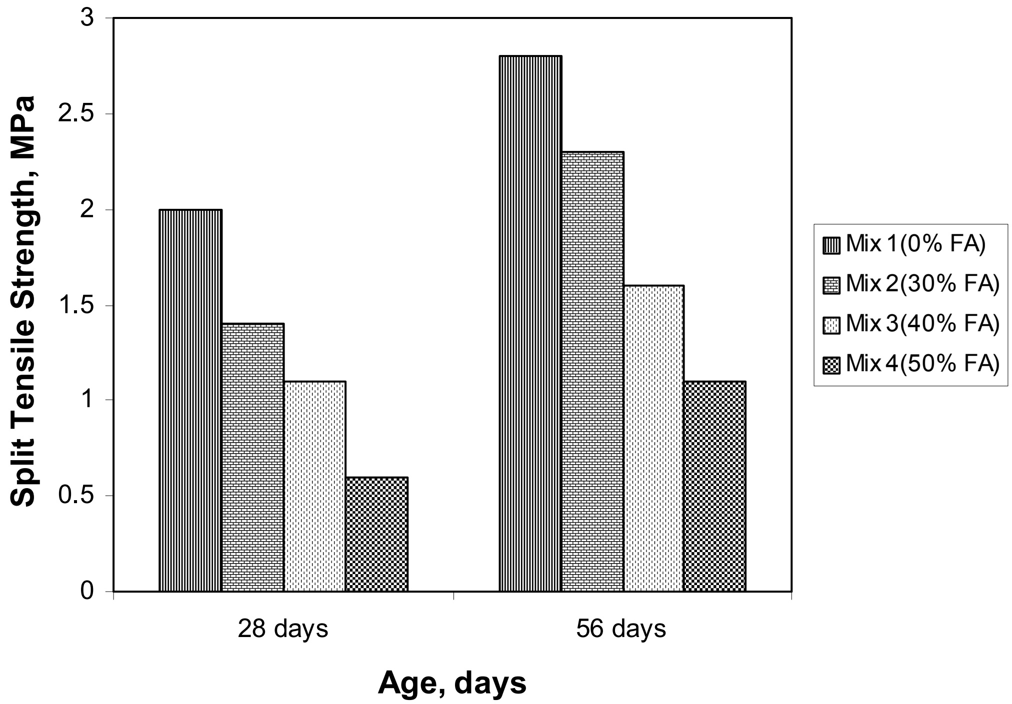

Fig. 4.18 Split Tensile Strength vs Age (at 80˚C)

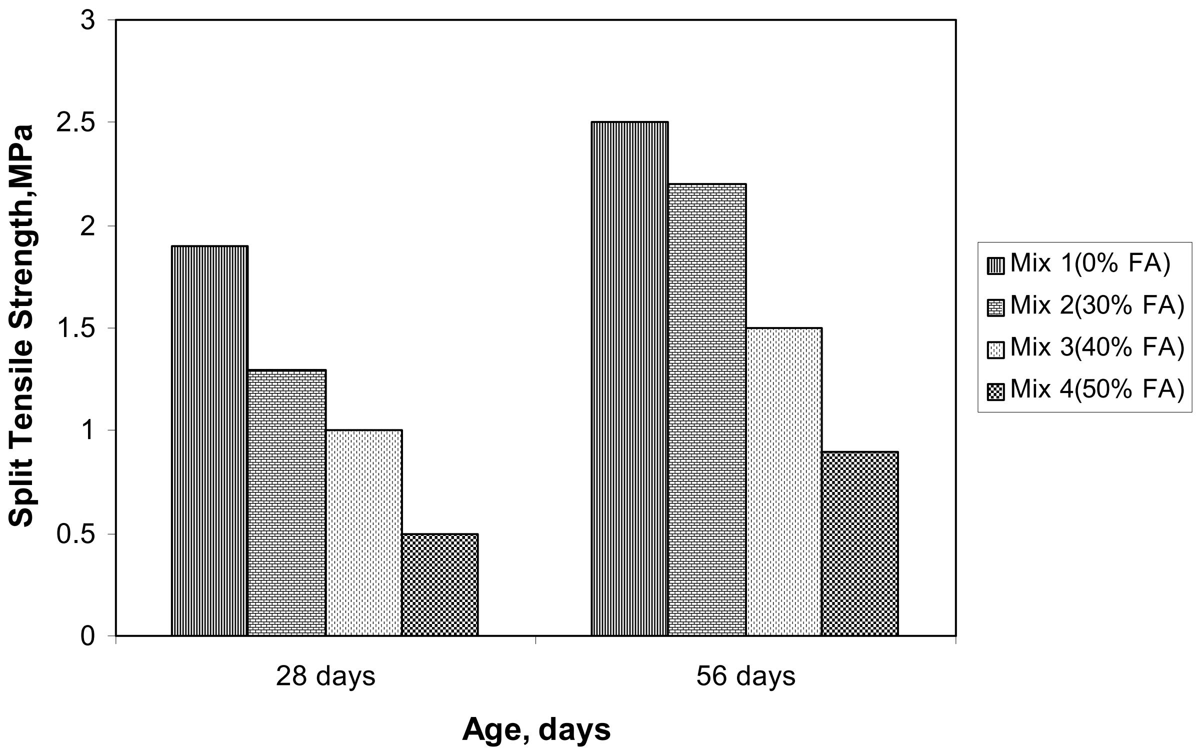

Fig. 4.19 Split Tensile Strength vs Age (at 100˚C)

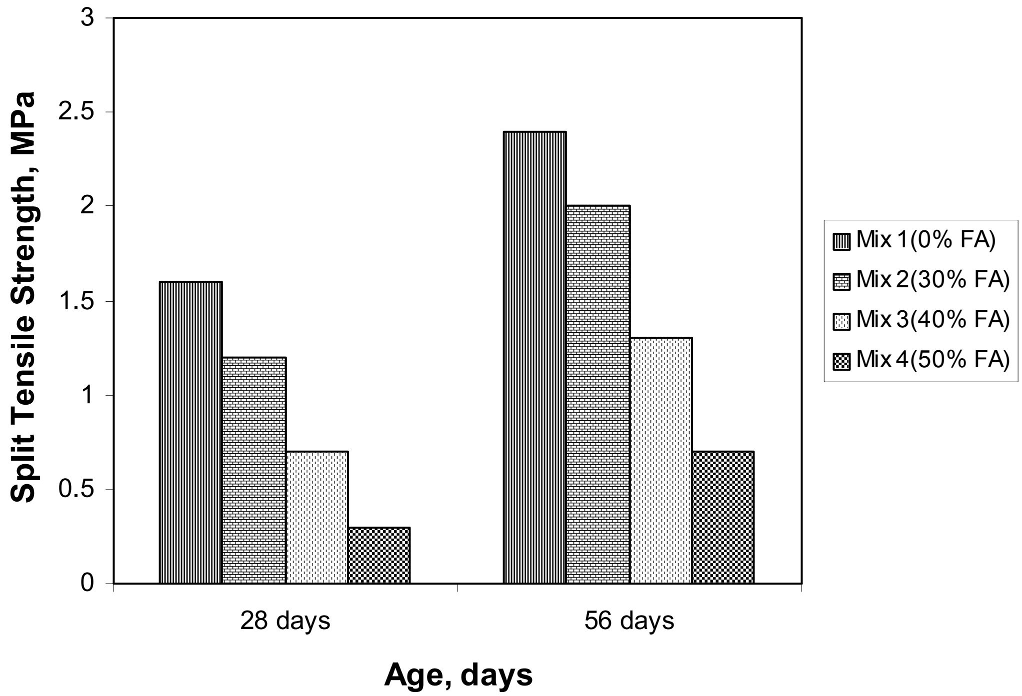

Fig. 4.20 Split Tensile Strength vs Age (at 120˚C)

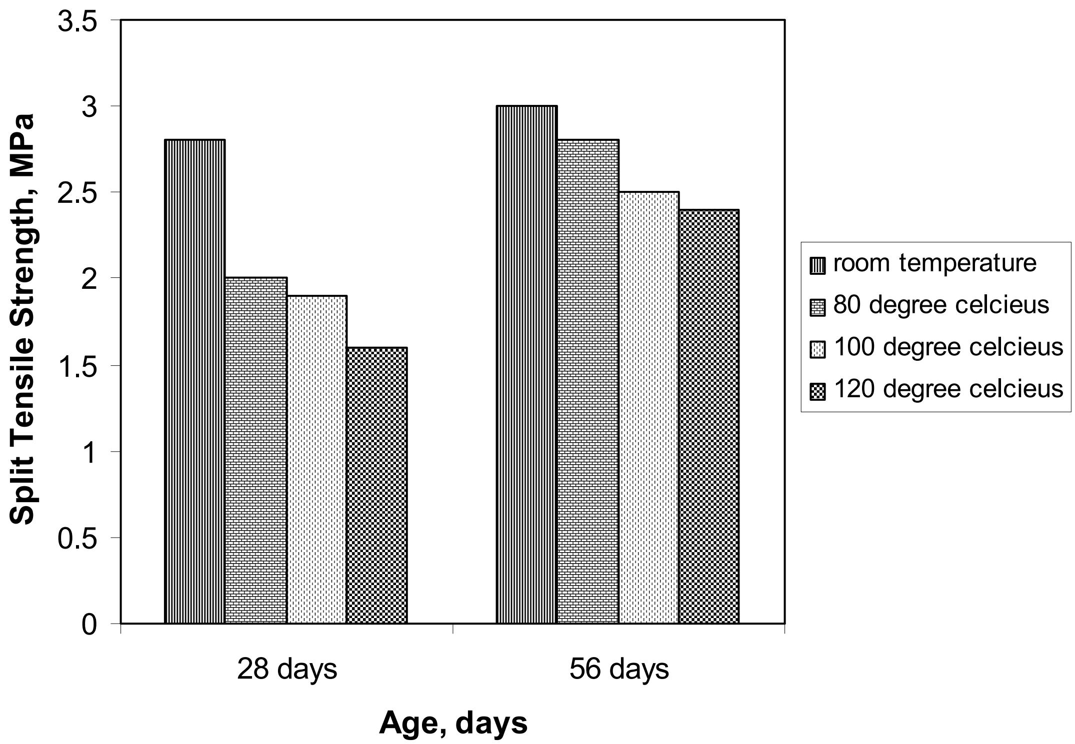

Fig. 4.21 Split Tensile Strength vs Age (without Fly ash)

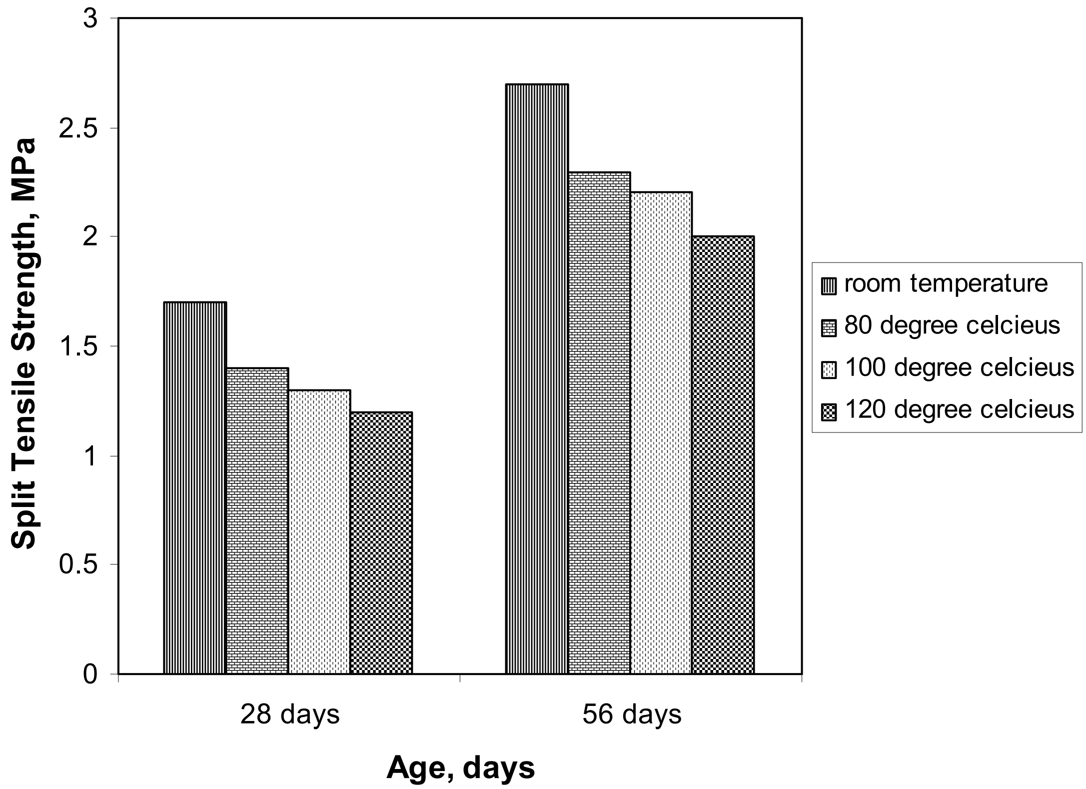

Fig. 4.22 Split Tensile Strength vs Age (with 30% Fly ash)

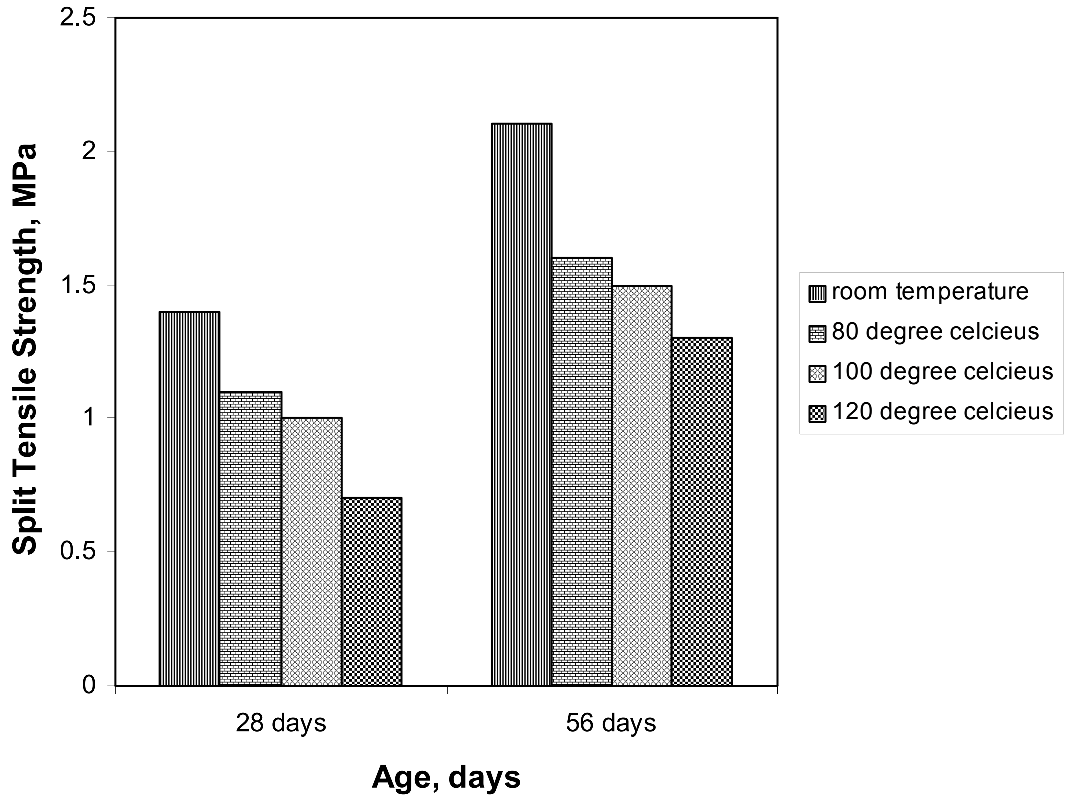

Fig. 4.23 Split Tensile Strength vs Age (with 40% Fly ash)

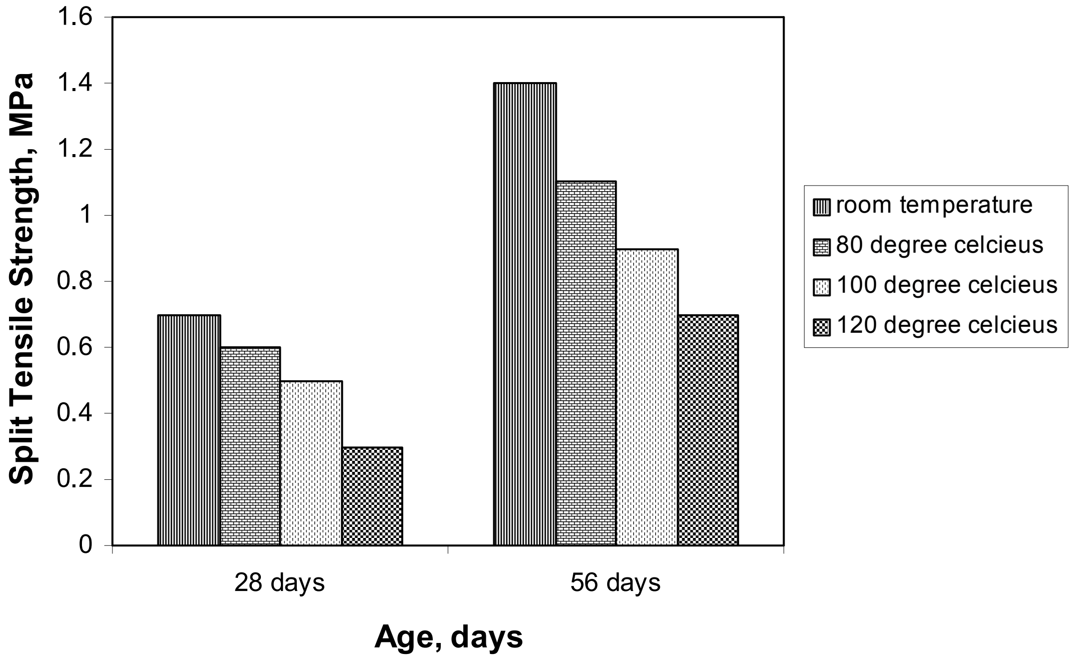

Fig. 4.24 Split Tensile Strength vs Age (with 50% Fly ash)

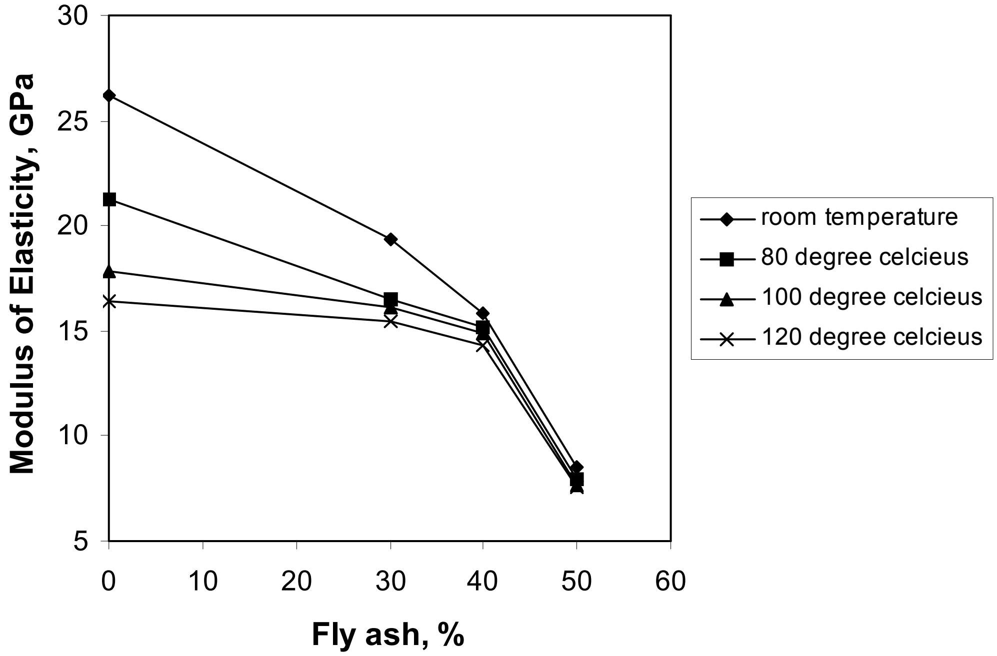

Fig. 4.25 Modulus of Elasticity vs Fly ash (56 days)

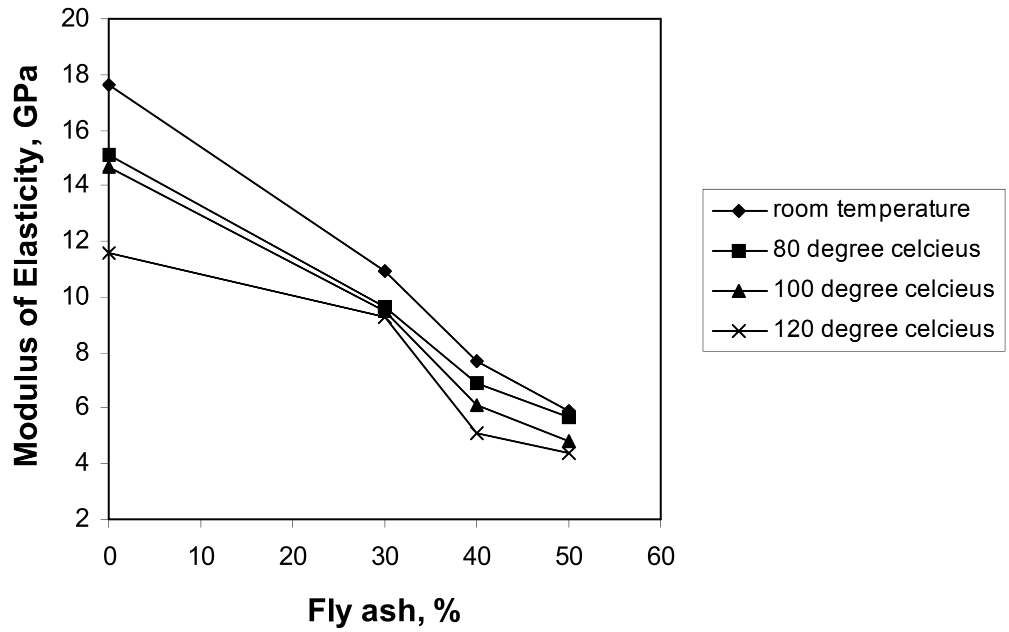

Fig. 4.26 Modulus of Elasticity vs Fly ash (28 days)

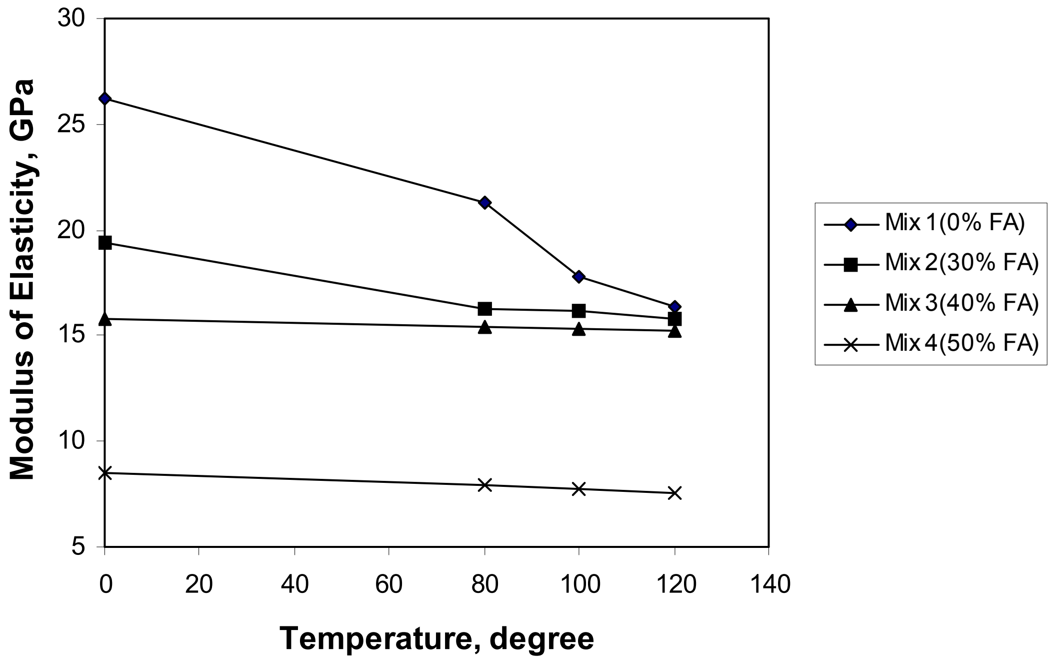

Fig. 4.27 Modulus of Elasticity vs Temperature (56 days)

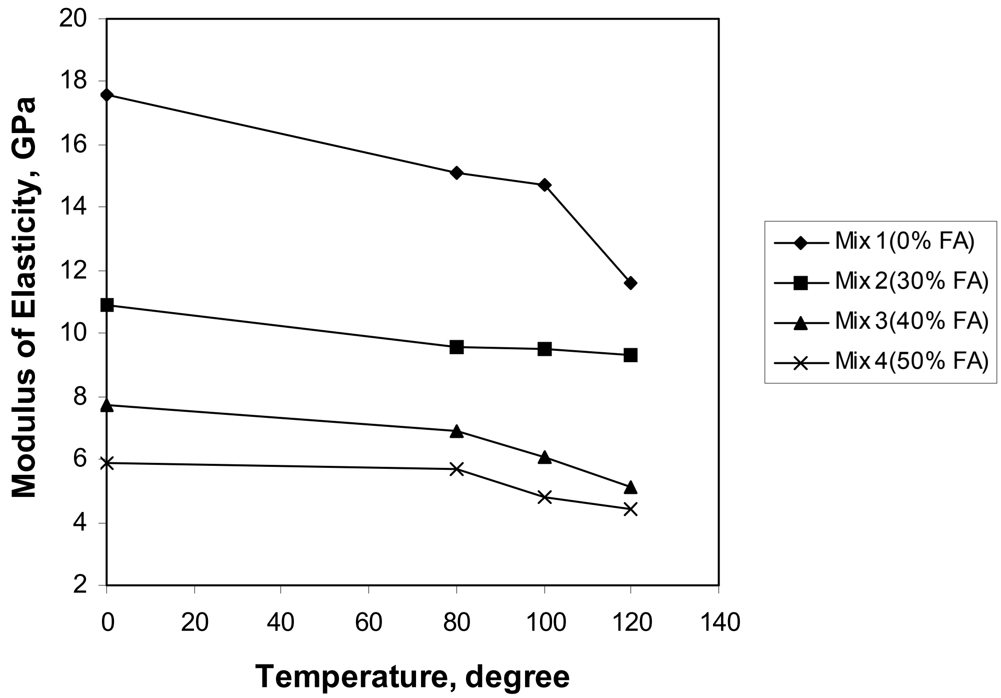

Fig. 4.28 Modulus of Elasticity vs Temperature (28 days)

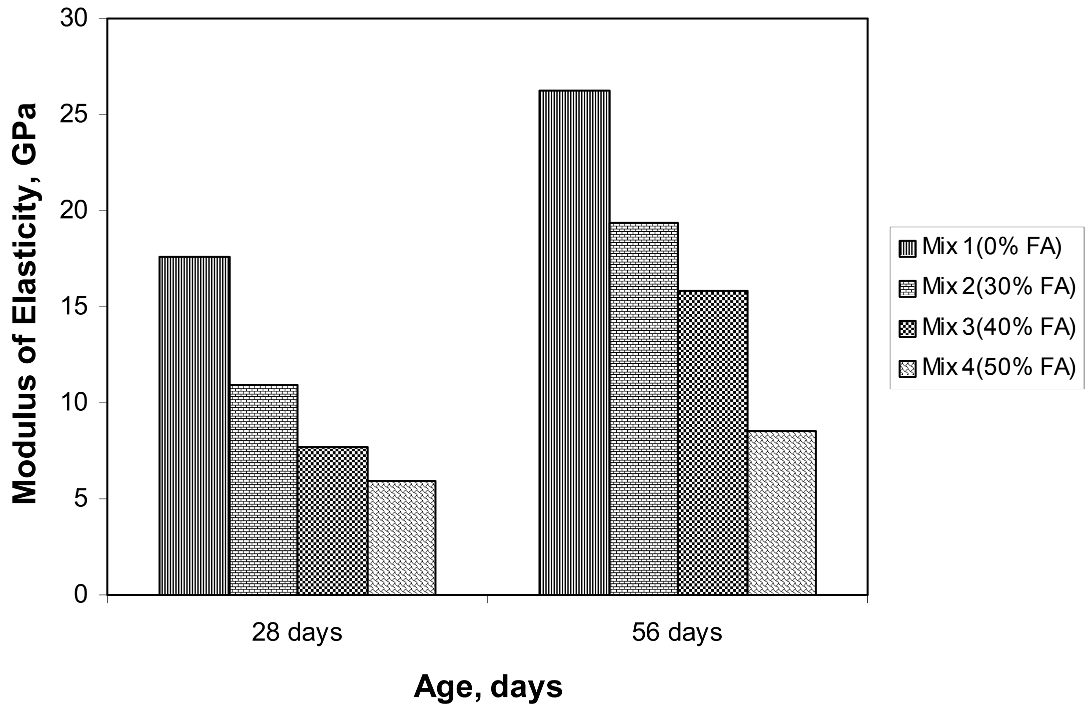

Fig. 4.29 Modulus of Elasticity vs Age (at room temperature)

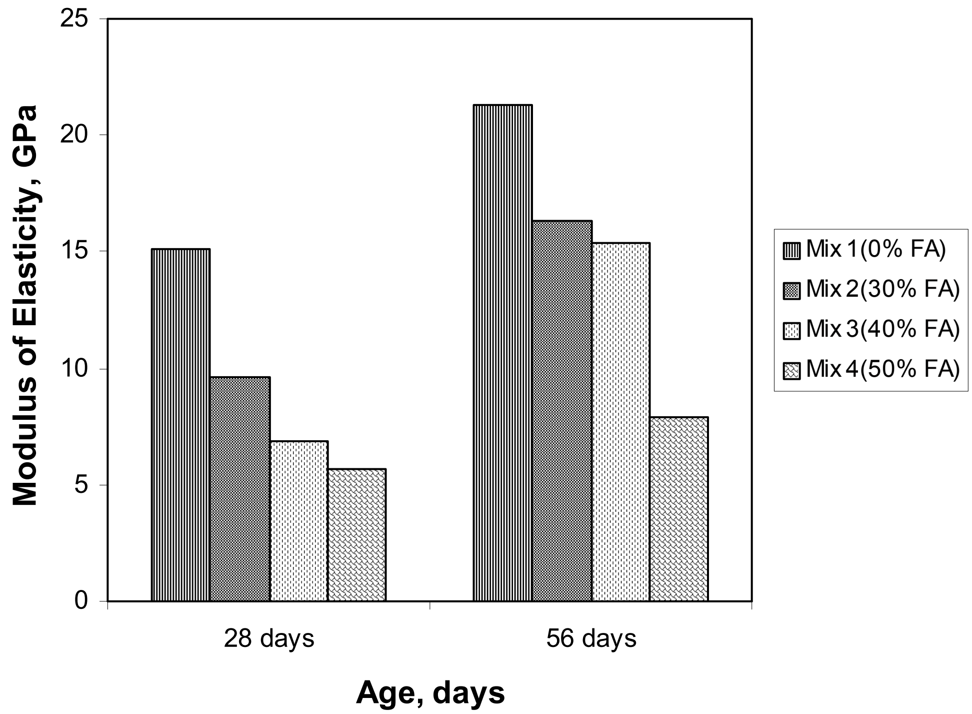

Fig. 4.30 Modulus of Elasticity vs Age (at 80˚C)

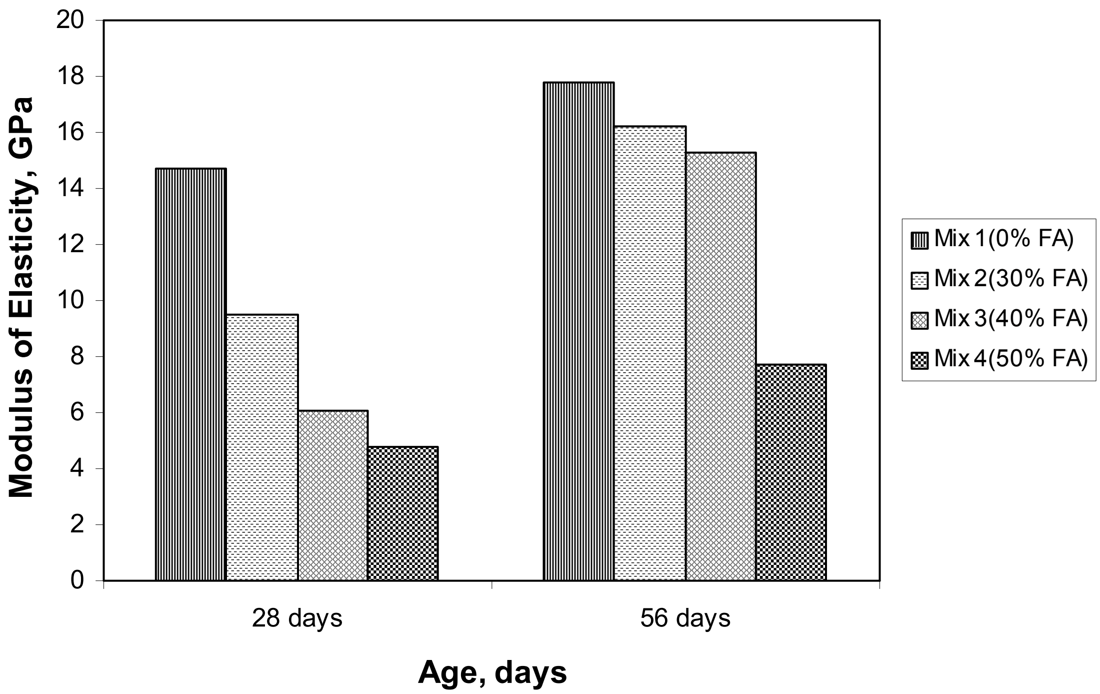

Fig. 4.31 Modulus of Elasticity vs Age (at 100˚C)

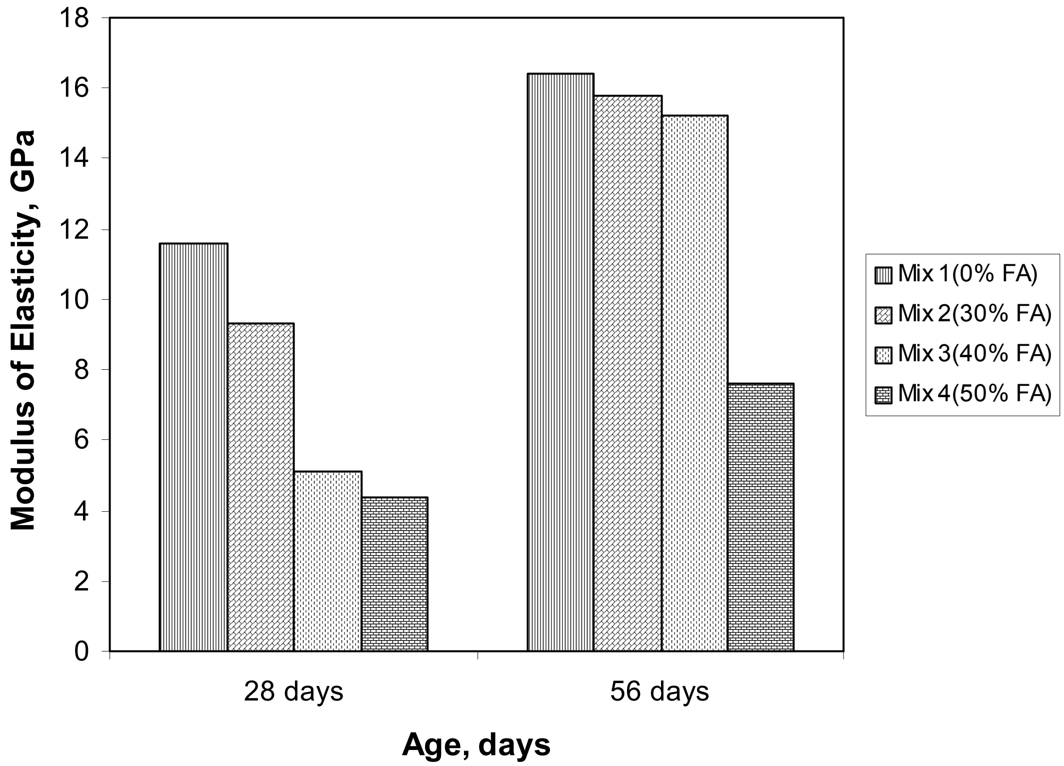

Fig. 4.32 Modulus of Elasticity vs Age (at 120˚C)

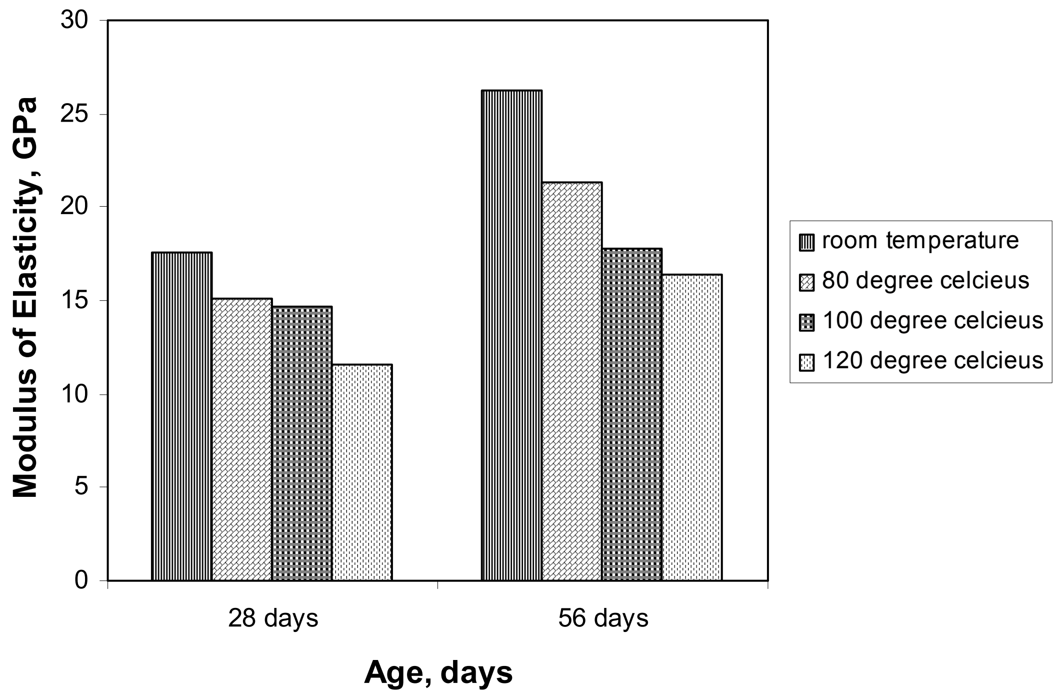

Fig. 4.33 Modulus of Elasticity vs Age (without Fly ash)

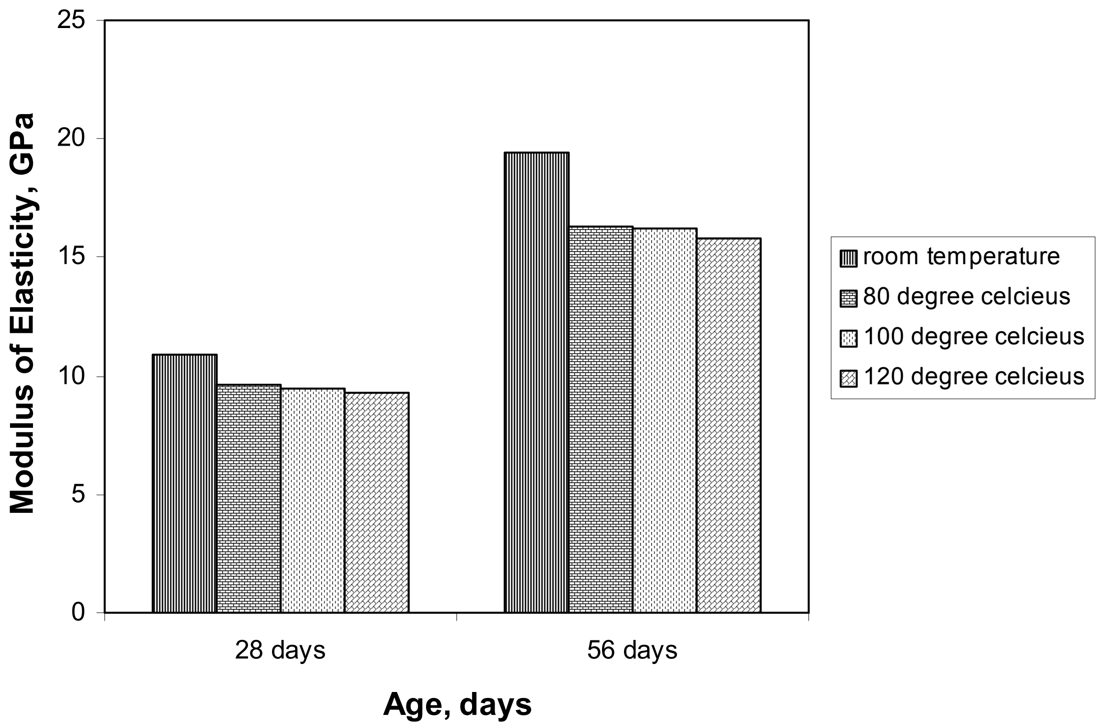

Fig. 4.34 Modulus of Elasticity vs Age (with 30% Fly ash)

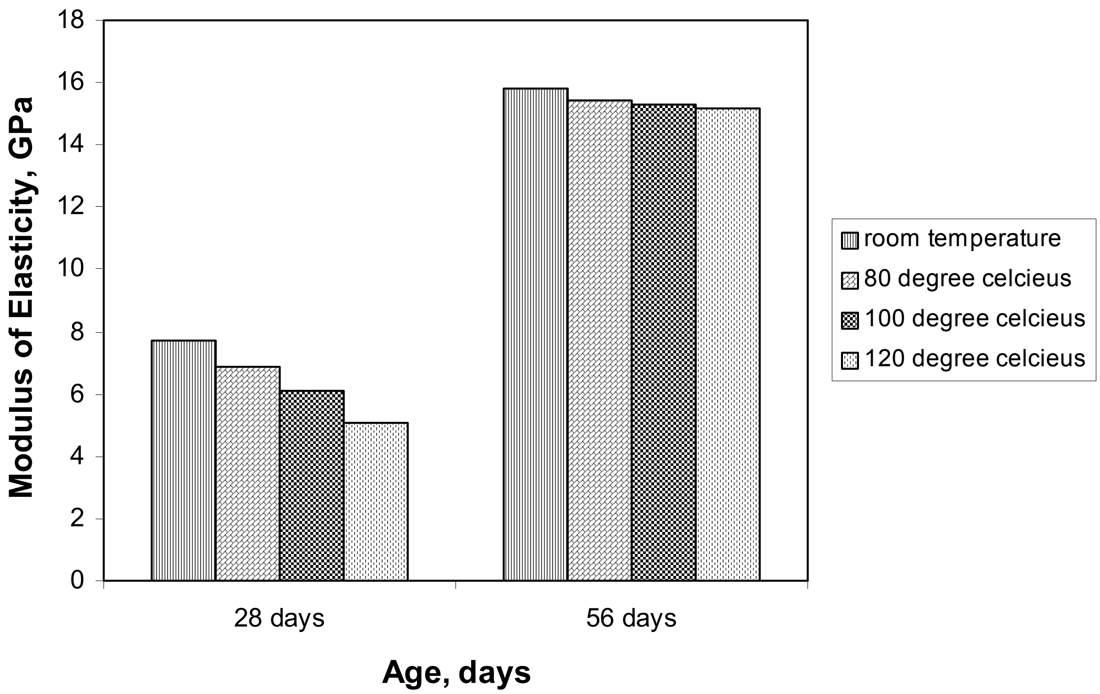

Fig. 4.35 Modulus of Elasticity vs Age (with 40% Fly ash)

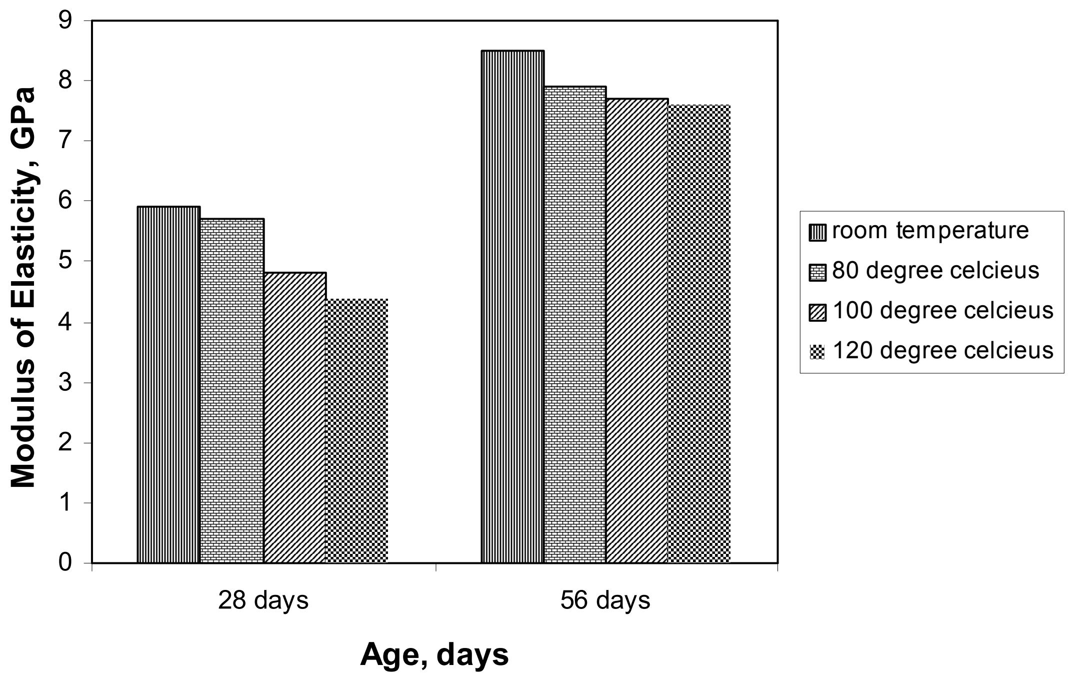

Fig. 4.36 Modulus of Elasticity vs Age (with 50% Fly ash)

CHAPTER- 5

CONCLUSIONS

The following conclusions are drawn from this study:

1. Compressive strength of concrete decreased with the increase in cement replacement with Class-F fly ash. However, at each replacement level of cement with fly ash, an increase in strength was observed with the increase in age.
2. With the variation of temperature compressive strength changed. With the rise in temperature from room temperature to 120˚C, compressive strength decreased.
3. Splitting tensile strength and modulus of elasticity increased with increase in age at each replacement level of cement with fly ash up to 50% but they were decreased with increase in volume of fly ash.
4. Increase in temperature up to 120˚C decreased the splitting tensile strength and modulus of elasticity, this is due to the chemical transformation of the gel weakened the matrix bonding, which brought about a loss of strength of fly ash concrete at high temperatures.
5. The specimens failed after the formation of a number of longitudinal (vertical) cracks in the loading direction, and no shear type failures occurred.

Picture 1: Failure of cylinder having no fly ash at room temperature (28 days)

Picture 2: Failure of cylinder having no fly ash at 80°C (28 days)

Picture 3: Failure of cylinder having no fly ash at 100°C (28 days)

Picture 4: Failure of cylinder having no fly ash at 120°C (28 days)

Picture 5: Failure of cylinder having 40% fly ash at room temperature (56 days)

Picture 6: Failure of cylinder having 40% fly ash at 80°C (56 days)

Picture 7: Failure of cylinder having 40% fly ash at 100°C (56 days)

Picture 8: Failure of cylinder having 40% fly ash at 120°C (56 days)

Picture 9: Splitting Tensile Failure of cylinder having 40% fly ash at room temperature (56 days)

Picture 10: Splitting Tensile Failure of cylinder having 40% fly ash at 80°C (56 days)

Picture 11: Splitting Tensile Failure of cylinder having 40% fly ash at 100°C (56 days)

Picture 12: Splitting Tensile Failure of cylinder having 40% fly ash at 120°C (56 days)

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