BEHAVIOUR OF LATERISED NORMAL AND SELF COMPACTING CONCRETE SUBJECTED TO ELEVATED TEMPERATURES

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Certificate

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DECLARATION

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ABSTRACT

Concrete is a universal material in the construction industry. With natural resources like sand and aggregate, fast depleting, it is time to look for alternate materials to substitute these in the process of making concrete. There are instances like exposure to solar radiation, fire, furnaces, and nuclear reactor vessels, special applications like missile launching pads etc., where concrete is exposed to temperature variations. Concrete is generally believed to be a good insulating material against temperature, However, when concrete is exposed to high temperature the transformations and reactions within the concrete cause progressive breakdown of cement gel structure and consequent loss in load bearing capacity, integrity and insulation capacity. With the increasing incidence of fire occurrence in modern structures, fire protection measures at the very design and construction stages of such structures has become inevitable. Fire resistance of concrete is affected by factors like the type of aggregate and cement used in its composition, the temperature and duration of the fire, sizes of structural members, moisture content of concrete etc. Since performance is more important than strength when exposed to fire, marginal materials could be effectively used for making concrete that resist fire exposure. One of the potential marginal materials that can be used in concrete is laterite. Laterite is abundantly available in many parts of the world, but the use of laterite in the making of concrete is not fully utilised so far.

Use of fly ash and Ground Granulated Blast Furnace Slag (GGBFS) in concrete, not only improves the rheological properties, but also reduces pollution to the environment. Self Compacting Concrete (SCC), which is one of the most outstanding advancements in concrete technology could be effectively used in jacketing of structural members, repair and retrofitting etc.

A technology for making SCC incorporating marginal materials and supplementary cementitious materials could lead to an economic and environment friendly material to be used in places where strength is not a primary criteria.

In this research work, an attempt has been made to understand the behaviour of concrete when weathered laterite aggregate is used in both conventional and self compacting normal strength concrete. The study has been extended to understand the thermal behaviour of both types of laterised concretes and to check suitability as a fire protection material.

A systematic study of laterised concrete considering parameters like source of laterite aggregate, grades of Ordinary Portland Cement (OPC) and types of supplementary cementitious materials (fly ash and GGBFS) has been carried out to arrive at a feasible combination of various ingredients in laterised concrete.

A mix design methodology has been proposed for making normal strength laterised self compacting concrete based on trial mixes and the same has also been validated.

The physical and mechanical properties of laterised concretes have been studied with respect to different variables like exposure temperature (200°C, 400°C and 600°C) and cooling environment (air cooled and water cooled).

The behaviour of ferrocement elements with laterised self compacting concrete has also been studied by varying the cover to mesh reinforcement (10mm to 50mm at an interval of 10mm), exposure temperature and cooling environment.

Based on the present study, it has been observed that the compressive strength of concrete with weathered laterite all-in aggregate is lower compared to a corresponding conventional concrete. It has been found here that a 9% lower strength has been observed for laterised concrete when compared to the strength of M25 grade conventional concrete.

It has also been found that a 20% replacement of OPC by fly ash or a 25% replacement of the same by GGBFS yield economic concrete with no significant loss in compressive strength.

Unlike conventional SCC, laterised self compacting concrete requires large quantity of additions (fly ash or GGBFS) to achieve required flow properties.

The loss of unit mass of laterised concrete when exposed to a temperature level between 200°C and 400°C is not significant and is attributed to the physically adsorbed water. However, for exposure temperature above 400°C, considerable reduction in unit mass has been observed and is attributed to the loss of chemically combined water present in hydrated cement products.

When mineral admixture (fly ash or GGBFS) was added, the conventional concrete did not crack up to 600°C and laterised concrete did not develop any crack even at 800°C. However laterised self compacting concrete developed distributed hair line cracks at 600°C.

The loss of chemically combined water in concrete is one of the major factors that control the cracking of concrete when exposed to high temperature.

In conclusion, the combined use of weathered laterite aggregate and additions as fly ash or GGBFS in laterised concrete (LC) and laterised self compacting concrete (LSCC) form green and economical concrete which has better physical properties compared to conventional concrete when exposed to high temperatures. Hence these LC and LSCC are suitable as a fire protection material compared to conventional concrete.

ABBREVIATIONS AND NOMENCLATURE

ACI	-	American Concrete Institute
AD	-	Anno Domini
ASTM	-	American Society for Testing and Materials
BASF	-	Registered Trade Mark of Construction Chemicals (BADISCHE ANILIN-UNO SODA FABRIK)
CANMET	-	Canada Centre for Mineral and Energy Technology
CC	-	Control Concrete
CCFL	-	Control Concrete with Fly ash as Partial Replacement of Cement.
CCGG	-	Control Concrete with GGBFS as Partial Replacement of Cement
CC33	-	Control Concrete with 33 Grade OPC
CC43	-	Control Concrete with 43 Grade OPC
CC53	-	Control Concrete with 53 Grade OPC
CEB	-	Central Engineering Building.
CESS	-	Centre for Earth Science and Studies.
CSH	-	Calcium Silicate Hydrate.
CUSAT	-	Cochin University of Science and Technology.
C_w	-	Weight of Cement for $1m^3$ of Concrete (kg).
DPT	-	Double Punch Test.
EFNARC	-	European Federation for Specialist Construction Chemicals and Concrete Systems.
E_c	-	Elastic Modulus of Concrete at Room Temperature (MPa).
E_T	-	Elastic Modulus of Concrete after Exposure to T Degree Celsius (MPa).
FA	-	Fly ash.

F_a	-	Additions Factor.
F_w	-	Weight of Addition for $1m^3$ of Concrete (kg)
f_b	-	Modulus of Rupture at Room Temperature (MPa).
f_{bT}	-	Modulus of Rupture after Exposed to T °C (MPa).
f_{ck}	-	Cube Compressive Strength at 28 Days of Curing (MPa).
$\bar{f}_{ck}$	-	Target Mean Strength (MPa).
f_{ct}	-	Compressive Strength at an Age of 't' Days.
f_{cT}	-	Concrete Compressive Strength after Exposure to T Degree Celsius (MPa).
f_{cy}	-	Cylinder Compressive Srength at 28 Days of Curing (MPa).
f_s	-	Split Tensile Strength at 28 Days of Curing (MPa).
f_T	-	Tensile Strength of Concrete at Room Temperature (MPa).
f_{TT}	-	Tensile Strength of Concrete after Exposure to T Degree Celsius (MPa).
f_t	-	Flexural Tensile Strength at 28 Days of Curing (MPa).
GGBFS	-	Ground Granulated Blast Furnace Slag.
HPC	-	High Performance Concrete.
HVFA	-	High Volume Fly ash.
k	-	Statistical Constant.
LC	-	Laterised Concrete.
LCA	-	Laterised Concrete Prepared with Laterite All-in Aggregate.
LCAC33	-	LCA with 33 Grade OPC.
LCAC43	-	LCA with 43 Grade OPC.
LCAC53	-	LCA with 53 Grade OPC.

LCF	-	Laterised Concrete Prepared by Replacing Fine Aggregate with Laterised Fine Aggregate.
LCFL	-	Laterised Concrete with Fly ash as Partial Replacement.
LCGG	-	Laterised Concrete with GGBFS as Partial Replacement.
LSCC	-	Laterised Self Compacting Concrete
LSCCF	-	Laterised Self compacting Concrete with Fly ash as Addition.
LSCCG	-	Laterised Self Compacting Concrete with GGBFS as Addition.
M_f	-	Modification Factor.
M20	-	Design Concrete Mix with f_{ck} = 20 MPa.
M25	-	Design Concrete Mix with f_{ck} = 25 MPa.
M40	-	Design Concrete Mix with f_{ck} = 40 MPa.
min.	-	minutes
NC	-	Normal Vibrated Concrete.
OPC	-	Ordinary Portland Cement,
PCE	-	Polycarboxylic Ether.
PA2	-	Passing Ability Class-2.
SCC	-	Self Compacting Concrete .
SD	-	Standard Deviation.
SF2	-	Slump-Flow Class-2.
SFE1	-	Ferrocement element made with LSCCF with 10mm cover to mesh on all sides.
SFE2	-	Ferrocement element made with LSCCF with 20mm cover to mesh on all sides.

SFE3	-	Ferrocement element made with LSCCF with 30mm cover to mesh on all sides.
SFE4	-	Ferrocement element made with LSCCF with 40mm cover to mesh on all sides.
SFE5	-	Ferrocement element made with LSCCF with 50mm cover to mesh on all sides.
SNF	-	Sulphonated Naphthalene Formaldehyde
SP	-	Superplasticiser.
s	-	Standard Deviation
S_a	-	Specific Gravity of Addition (Fly ash or GGBFS)
S_c	-	Specific Gravity of Cement.
S_l	-	Specific Gravity of Laterite Aggregate.
T	-	Temperature of Fire in Degree Celsius ($\geq$ 20°C).
T_0	-	Initial Furnace Temperature (°C).
T_1	-	Furnace Temperature at Time t_1 (°C).
t	-	Curing Age of Concrete in Days.
t_1	-	Furnace Heating Time (minutes).
USA	-	United State of America.
VMA	-	Viscosity Modifying Admixtures.
VS2/ VF2	-	Viscosity Classification-2.
V_a	-	Volume of Air in Concrete.
W/P	-	Water Powder Ratio.
W_w	-	Weight of Water in (kg).
W_l	-	Weight of Laterite Aggregate (kg).

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Chapter-1
INTRODUCTION

One of the basic infrastructural facilities that man needs for good living is shelter. The development of technology in materials and construction has made it possible to build even skyscrapers. However, the increasing cost of conventional construction materials has made it difficult to meet the shelter requirements of the teeming population of developing countries.

Fast expansion of the construction industry brought forth with it associated problems. The most widely used construction material is concrete, commonly made by mixing portland cement with sand, crushed rock and water. Man uses no material except water in such huge quantity. The conventional fine aggregate used in concrete is river sand. This is fast becoming a rare and expensive commodity. Uncontrolled sand mining from river beds leads to problems like bank erosion lowering of water table and other adverse effects to the environment. Similarly, quarrying of granite is the main source of coarse aggregate and that also causes environmental issues. It is high time to think about an alternative to the aggregates. The necessity of using locally available materials (marginal materials) for the production of concrete is the need of the hour, particularly in fast developing countries like India.

Laterite is one such marginal material abundantly available in many parts of the world, particularly in tropics and sub tropics. In India, there are large deposits of laterite in the peninsular region, which have not been fully utilised so far.

Laterite is the product of intensive and long lasting tropical rock weathering which is intensified by high rain falls and elevated temperature. Laterite consists mainly of the minerals kaolinite, goethite, hematite and gibbsite which form in the course of weathering. Moreover, many laterite deposits contain quartz as relatively stable relic mineral from the parent rock. The iron oxides goethite and hematite cause the red brown colour of the laterite.

Laterised Concrete (LC) can be defined as the concrete in which part or all the fine and course aggregates are replaced by laterite aggregate. The utilisation of laterised concrete as a construction material has not yet become popular due to lack of proper understanding about its behaviour. Even though studies on the use of lateritic soil in concrete has been carried out in countries like Australia, Nigeria, the U S and India, they are limited to either the replacement of sand in concrete or its use as sub grade material in road construction. No systematic study has been reported on the suitability of laterite concrete as an alternative to conventional concrete, especially with laterite coarse aggregate.

Industrial by-products, such as fly ash and slag, invariably contain small quantities of toxic metals. The practice of using these for land filling, dumping into streams and ponds or even stockpiling, presents serious health hazards. However, when utilised in blended portland cements or as mineral admixtures in concrete, the toxic metals become immobilised in the form of insoluble products of cement hydration and thus are rendered harmless. Also these industrial by-products which are generally pozzolanic or cementitious serve as supplementary cementing materials and enhance durability and other engineering properties of portland cement concrete products. Combined use of laterite aggregate and supplementary cementitious materials may lead to an economical concrete.

Self Compacting Concrete (SCC) is an innovative concrete made with same materials of normal concrete that does not require vibration for compaction. It flows under its own weight; completely filling formwork and achieves full compaction, even in the presence of congested reinforcement. The hardened concrete is dense homogeneous and has even better engineering properties and durability compared to traditional concrete.

The type of aggregate used has influence on spalling of concrete when exposed to fire (high temperature). Marginal materials like laterite might perform better because of their better thermal stability.

Hence a comprehensive study on the physical and mechanical properties of laterised concrete is required to confirm the suitability of laterised concrete for construction purpose.

In cases where compaction and placing are difficult, such as jacketing of structural steel element for fire protection, casting of ferrocement element etc., self compacting concrete is a better choice than conventional concrete. Most of the existing studies in this area are focused on the strength behaviour of concrete subjected to elevated temperature but not in line with it as a fire protection material. Apart from mechanical properties, formation of cracks, its pattern, spalling of cover concrete, colour changes etc., should also be considered while using concrete as a fire protection material. Only limited information is available with regard to the thermal properties of laterised concrete (LC) and Laterised Self Compacting Concrete (LSCC) which are subjected to elevated temperatures.

Ferrocement is a form of reinforced concrete using closely spaced multiple layers of mesh or small diameter rods completely infiltrated with and encapsulated in mortar. The most common type of reinforcement is steel mesh.

Other materials selected- organic, natural, or synthetic fibers may be combined with metallic mesh. Applications of ferrocement are numerous, especially in structures or structural components where self-help or low levels of skills are required. Besides boats and marine structures, ferrocement is used for housing units, water tanks, roofing sheets etc..

In the present investigation, it is proposed to conduct a systematic study to check the suitability of laterite aggregate for preparing concrete and self compacting concrete and the performance of such concrete when exposed to elevated temperatures. This investigation includes behavioral study of concrete with different types of cements, supplementary cementitious materials (fly ash and ground granulated blast furnace slag), development of mix design methodology for laterised self compacting concrete, shear strength parameters and behaviour of ferrocement element made with laterised self-compacting concrete.

The contents of various chapters of this thesis are briefly described below.

Chapter 1 is the introductory chapter and makes general observations on the need of the present research work and the highlights of the present study. A brief out line of each chapter is also presented here.

Chapter 2 presents a review of the investigations carried out by earlier workers. Details of laterite aggregates, influence of types of cements and supplementary cementitious materials in concrete, behaviour of concrete, self compacting concrete, ferrocement and influence of temperatures on concrete are the key areas of review. A critical discussion has been presented based on the review of literature specific to area of study. Scope and objectives of the present study have been derived based on the above and the same is presented in this chapter.

Chapter 3 deals with details about various materials used for the present study and their test results. The methods of testing self compacting concrete, heating of specimen etc. are also mentioned briefly in this chapter. An experimental setup for the determination of shear strength (Mode II fracture) of concrete has been fabricated and its details along with test procedure have been presented in this chapter.

Chapter 4 deals with the preliminary study on the suitability of laterite aggregate in concrete. Laterised concrete has been developed either by replacing fine aggregate or by replacing both fine and coarse aggregates in conventional concrete. The properties of laterised concrete at fresh and hardened stages have been compared with the corresponding properties of conventional concrete. To study the influence of the source of laterite aggregate, concrete has been made using laterite aggregate collected from various sources and their properties were compared. The influence of type of cement and supplementary cementitious materials in concrete has also been discussed in this chapter.

Chapter 5 proposes a mix design methodology developed by this researcher for laterised self compacting normal strength concrete (M20 to M40 grade) and provides details about the experimental studies carried out to validate the proposed mix design methodology.

Chapter 6 discusses the influence of various parameters on the physical and mechanical properties of laterised concrete when exposed to high temperature. The laterised concrete specimens, both vibrated and self compacting type, were heated to different temperature levels (200ºC, 400ºCand

600°C). The specimens were then cooled to ambient temperature in two different ways, namely air cooling and water cooling. The shear strength (mode II), influence of temperatures on strengths of concrete, modulus of elasticity, cracking behaviour, colour variation have been discussed in this chapter. The self compacting laterised concrete flexural ferrocement elements at elevated temperatures have also been discussed in this chapter in detail.

Chapter 7 presents the summary of the work carried out and major conclusions derived based on the detailed study and its discussion. Suggestions for further study in the related area have also been given in this chapter.

Chapter-2

REVIEW OF LITERATURE

2.1 Introduction

With the fast depleting state of natural resources like river sand and aggregates, it is time to look for alternative cheap materials (marginal materials) for making concrete, particularly when strength is not a primary parameter.

Fire remains one the most serious potential risks to buildings, especially for industrial structures made with steel. Most structural materials are affected when exposed to high temperature. One of the methods for protecting steel against fire is by encasing it with concrete (Jacketing). Such concrete should perform its required function against fire and generally strength is not a governing criteria.

One of the potential marginal materials for use in concrete is laterite. A ferruginous, vascular, soft material occurring within the soil which hardens irreversibly on exposure to weather and used as a building material, was first recognised as 'laterite' by Francis Hamilton Buchanan (1902-1929) a medical officer. He suggested the name laterite, from *later*, the Latin word for brick [1].

2.2 Genesis of Laterite

Laterite is a product of intense sub aerial weathering. Laterisation process involves leaching of alkalis, basis and silica with complimentary enrichment of alumina, iron and some trace elements. This type of weathering advances for a faster degree in tropical regions where the temperatures and seasonal rainfall is the highest, giving rise to alternate wet and dry conditions. Invariably, all the rock types under these conditions give rise to laterite, which look similar in appearance. However, there is a pronounced change in mineralogical and migration of elements in laterite profiles.

The first global synthesis of the distribution of laterite was done by Prescott and Pendleton in 1952 [2]. Laterite and associated soils are widely distributed in the tropics and subtropics of Africa, Australia, India, South-East Asia and South America. Figure 2.1 shows the pictorial representation of laterite deposit world over [2].

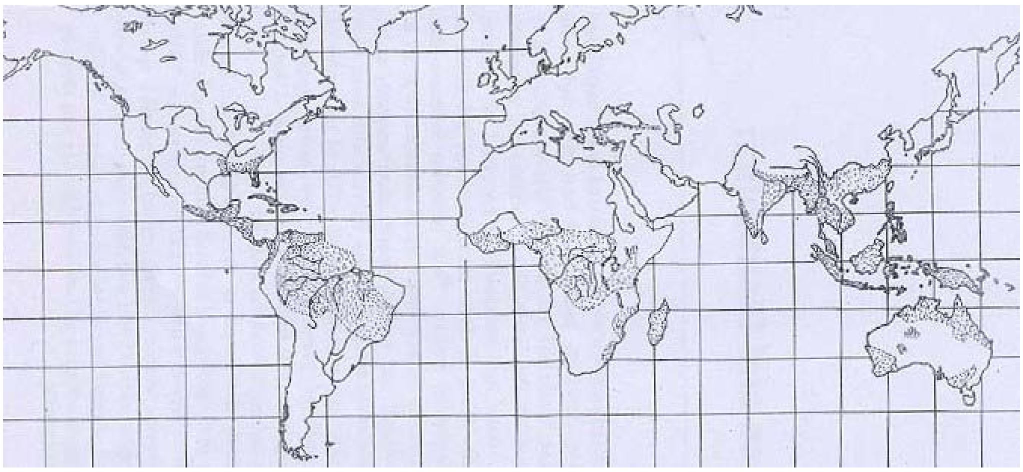

Figure 2.1 Distribution of laterites and associated soils in the tropics and subtropics[2]

In India, laterite soils occupy an area of about 1,30,066 Sq.km and is well developed on the summits of Deccan hills, Karnataka, Kerala, the Eastern Ghats, West Maharashtra and central parts of Orissa and Assam. The laterite terrain of Kerala occupies the midland region of the state and covers about 60% of the state. Mature laterite is made up primarily of iron, aluminum, silica, titanium and water. Generally laterite is poor in alkali and alkaline earth metals. The average chemical composition of typical laterite is presented in Table 2.1.

Table 2.1 Chemical composition of typical laterite.

Sl. No.	Chemical compound	Typical range in % of mass
1	H_2O	20 - 30
2	Al_2O_3	50 – 60
3	Fe_2O_3	35 – 80
4	SiO_2	Very low
5	TiO_2	About 2
6	Cr_2O_3	0 – 5.3
7	V_2O_5	0.01 – 0.65
8	Alkali and Alkaline Earths	Do not exceed 1

Laterite is being extensively used as building block from the early civilization. Some of the structures constructed during Khmer civilization in Cambodia (802AD to 1431AD) still stand virtually untouched by time [2]. The Fort at Bekal and Thalassery, Kerala constructed with laterite block ways back in 15^{th} century is still a standing monument [3]. Even today, laterite is used for construction of houses and other structures, for sub grade material in road construction etc. Present day increase in population and material consumption level warrants us to take stock of the renewable and nonrenewable resources needed for day to day needs and also for future needs.

2.3 Weathered Laterite

Latreite deposit is soft and capable of hardening on exposure to wetting and drying. The hardened laterite observed on the surface of laterite deposit has been referred to as ironstone or lateritic gravel [2]. The hardening is a complex phenomenon which includes two process viz. crystallisation and dehydration apart from gross or local enrichment. However iron has the key role in the hardening process. The weathering produces the laterite which is no longer a soil, but a consolidated material and can be mined or quarried. Weathered laterite usually appears as surface loamy soil with iron oxide pellets and gravels [2].

2.4 Laterised Concrete

The shortage of building materials coupled with the continuous increase in cost of procuring them are just two out of all the factors responsible for the current acute shortfall in the provision of adequate housing. Therefore the need for a research work aimed at reviewing the use of these materials or providing and finding alternative materials, but which are relatively cheap and available cannot be overemphasized. This effort would go a long way in alleviating the problem of shelter provision confirming the teeming population, especially of developing countries. To this end, intensive investigations have been on to develop and establish engineering basis for the use of lateritic soils, which are abundantly available, as substitute of aggregates in construction works, most especially concreting. In the light of this, studies that determine the proportions of concrete components that give optimum strength characteristics have been carried out.

The first published work on laterised concrete appears to have been by Adepegba in 1975 [4]. He compared the strength properties of normal concrete with those of concrete in which soft laterite as fine aggregate. Such concrete prepared by partial or full replacement of aggregate by laterite aggregate is called as laterised concrete. The laterised aggregate used for the works were collected from a borrow pit situated in Ilf- Ilewara, Nigeria. He observed that the plain laterite concrete is inferior to plain normal concrete as far as density and compressive strength is concerned. The impact resistance decreases with increases in percentage of laterite content in the concrete mix. Further, the modes of failure in the laterised concrete specimens are essentially the same as those in the plain concrete and they are brittle and occurred through the granite aggregate particles. He also observed that the flexural strength and workability of a laterised concrete mix of 2:3:6 with water/cement ratio of 0.65 by weight compared favorably with those of a normal concrete mix of 1:2:4 with water/ cement ratio of 0.65 by weight. Based on his study, he concluded that a concrete in which laterite fines are used in place of sand could be used as a structural material which is a substitute to the normal concrete.

Balogun and Adepegba [5] studied on the effect of varying sand content in laterised concrete and observed that, when sand is partially replaced with laterite fines, the most suitable mix for structural application is 1: 1.5: 3 with water cement ratio of 0.65 provided that the laterite content is kept below 50% of the total fine aggregate. They also observed that the modulus of elasticity of the recommended mix of laterised concrete (1: 1.5: 3) may be as high as 18-20 MPa if the mix is well controlled.

It has also been established from another study on effect of grain size on the strength characteristics of cement–stabilised lateritic soils by Lasisi and Ogunjide [6] that the higher the laterite/cement ratio, the lesser the compressive strength and that the finer the grain size range, the higher the compressive strength of cubes made from such soils. They have also reported that 10% cement by weight is needed to stabilise laterite soils to produce blocks of the same order of compressive strength as standard laterite blocks (450mm×225mmx150mm).

Osunade and Babalola [7] conducted studies on the effect of mix proportion and reinforcement size on the anchorage bond stress of laterised concrete and have established that both mix proportion and the size of reinforcement have significant effect on the anchorage bond stress of concrete made with laterite fine aggregate. The richer in terms of cement content in the mix proportion, the higher the anchorage bond stress of laterised concrete. Also, the anchorage bond stress between plain round steel reinforcement and laterised concrete increases with the increase in the size of reinforcement.

Falade [8] examined the influence of water to cement ratios and mix proportions on workability and strength of concrete containing laterite fine aggregate and observed that water requirement for a mix increases with increase in laterite to cement ratio and the strength decreases with increase in laterite to cement and water to cement ratios. It was further reported that the workability decreases with increase in laterite to cement ratio. Finally he has concluded that the well established variations of workability and compressive strength of normal concrete with water to cement ratios are valid for laterised concrete also.

The studies on impact resistance of plain laterised concrete with laterite fine aggregate by Oyekan and Balogun [9] found that the plain laterised concrete is inferior to normal concrete as far as compressive strength and impact resistance properties are concerned. Results also show that impact resistance of laterised concrete decreases as the laterite content increased in the cement matrix. Furthermore, at a constant water to cement ratio of 0.65, the standard mixes of 2:3:6 gives generally the highest impact resistance value.

Salau and Balogun [10] conducted investigation into the physical and strength properties, as well as shrinkage deformation characteristics, of laterised concrete with varying percentage of laterite content. They observed that there exists a consistent pattern of shrinkage-time curves for both normal and laterised concrete for both sealed and unsealed specimens. They have also reported that the shrinkage strain of laterised concrete is several times greater than that of normal concrete depending on the content of laterite in fine aggregate. Further they have observed that the instantaneous modulus of elasticity and compressive strength of laterised concrete with about 25% laterite content compare favorably with those of normal concrete of similar mix proportion and water to cement ratio by weight.

Osunade [11] studied the effect of replacement of lateritic soils with granite fines on the compressive and tensile strengths of laterised concrete. He observed that the maximum compressive strength values were obtained for laterite concrete containing 50% granite fines among the mix proportions considered (1:1:2, 1:1.5:3, 1:2:4 and 1:3:6). Also the addition of granite fines in laterised concrete resulted in a decrease in tensile strength. He concluded that laterised concrete containing granite fines can be used in the construction of buildings and rural infrastructures.

Udoeyo et al. [12] studied the strength performance of laterised concrete. They have observed that the workability of laterised concrete increases with increases in the replacement level of sand by laterite while the strength properties were increased with age but decreased with increase in replacement level of sand. They have proposed relationship for flexural strength and split tensile strength with the cube compressive strength as described by the regression equations (2.1) and (2.2).

$$f_t = 0.0939(f_{ck})^2 - 3.217f_{ck} + 32.216 \quad \text{.....(2.1)}$$

$$f_s = 0.013(f_{ck})^2 + 0.372\, f_{ck} + 5.187 \quad \text{.....(2.2)}$$

Ata [13] reported the effects of varying curing age and water to cement ratio on the elastic properties of laterised concrete, the Poisson's ratio of laterised concrete ranges between 0.25 and 0.35 and increases with age at a decreasing rate. Methods of curing, compaction and water to cement ratio have little influence on the Poisson's ratio. They observed an increase in Poisson's ratio for laterised concrete as the mix becomes lean.

Ata and Adesanya [14] studied the effects of applied stress on the modulus of elasticity and modulus of deformability (elastic and plastic) of laterised concrete. They concluded that the modulus of elasticity and modulus of deformability of laterised concrete deceases with an increase in level of the applied stress. They found both modulai increases with an increase in the strength of laterised concrete with time, but the increase in modulai are less than the corresponding increase in strength with time. They also found that the modulus of elasticity of laterised concrete is always higher than the corresponding modulus of deformability.

Oluwaseyi et al. [15] conducted studies on the weathering characteristics of laterised concrete with laterite to granite fine ratio as a factor. They observed that, for laterised concrete mix 1:2:4 and curing age at 28 days with varying laterite-granite fine ratio from 0 to 80 had reasonably high compressive strengths for temperature applications up to 125°C. When they exposed the same laterised concrete to alternate wetting and drying, compressive strength obtained were as low as 18 MPa. Therefore they concluded that laterite concrete depreciates with time under the prevailing conditions (rainy or dry) in the tropic. Optimum compressive strength could be obtained for a laterite grains to fine ratios between 40 and 60% at temperatures of 75-125°C.

Udeyo et al. [16] conducted studies on the influence of specimen geometry on the strengths of laterised concrete. They found that the specimen geometry had significant impact on the strength of laterised concrete.

The application of laterised concrete is limited to rural areas only mainly because of the lack of accepted standards of design parameters. From the literature survey it could be possible to understand that, the past researchers gave more emphasis to replace sand with laterite fine aggregates and got encouraging results, but no attempt has been reported about the judicious use of laterite coarse aggregate and all-in aggregate in concrete.

2.5 Mineral Admixtures

The use of pozzolanic material is as old as that of the art of concrete construction. It was recognised long time ago, that the suitable pozzolans used in appropriate amount, modify certain properties of fresh and hardened concrete.

Cement is the backbone for global infrastructural development. Production of every ton of cement emits carbon dioxide to the tune of about 0.87 ton. Because of the significant contribution to the environmental pollution and to the high consumption of natural resources like lime stone etc., there is a need to limit the use of cement. One of the practical solutions to reduce the use of cement is to replace cement with supplementary cementitious materials like pozzolana. It has been amply demonstrated that the best pozzolans in optimum proportions mixed with portland cement improves many properties of concrete namely,

- Reduced heat of hydration and thermal shrinkage.
- Increased the water tightness.
- Reduced the alkali-aggregate reaction.
- Improved resistance to attack by sulphate soils and sea water.
- Improved extensibility.
- Improved workability.

2.5.1 Pozzolanic Materials

Pozzolanic material are siliceous and aluminous materials which in themselves posses little or no cementitious value, but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide liberated on hydration of ordinary portland cement at ordinary temperature, to form compounds, possessing cementitious properties. During hydration of cement tri-calcium silicate, di-calcium silicate and calcium hydroxide etc. are formed. The siliceous or aluminous compound in a finely divided form react with the calcium hydroxide to form highly stable cementitious substances of complex composition involving water, calcium and silica. Generally amorphous silicate reacts much rapidly than the crystalline form. It is pointed out that calcium hydroxide; otherwise, a water soluble material is converted into insoluble cementitious material by the action of pozzolanic materials. Pozzolans can be grouped into natural and artificial. Natural pozzolans include,

- Clay and Shale
- Optline chert
- Diatomaceous earth
- Volcanic tuffs and Pumicities

Artificial pozzolans include

- Fly ash
- Blast furnace slag
- Silica fume
- Rice husk ash
- Metakaoline
- Surki

Most generally used pozzolanic materials in concrete are fly ash and blast furnace slag.

2.5.2 Fly Ash

Fly ash is a finely divided residue resulting from the combustion of powdered coal and transported by the flue gases and collected by electronic precipitator. Fly ash is the most widely used pozzolanic material all over the world. In the recent time, the importance and use of fly ash in concrete has grown so much that it has almost become a common ingredient in concrete, particularly for making high strength and high performance concrete. Extensive research has been done all over the word on the benefits that could be accrued in the utilisation of fly ash as a supplementary cementitious material [17, 18, 19 and 20].

The use of fly ash as concrete admixture not only extends technical advantages to the properties of concrete but also contributes to the environmental pollution control. India alone, produce about 75 million tons of fly ash per year, the disposal of which has become a serious environmental problem. The effective utilization of fly ash in concrete making is therefore attracting serious consideration of concrete technologists and government departments.

Apart from the technical advantages, other pressing factors, which demand a critical evaluation for increasing the use of fly ash in the production of concrete, are listed below.

- The growing dependence on coal for production of cement as a major source of fuel for electricity generation.
- The availability of fly ash as an industrial waste

- Need to protect the environment.
- Saving the energy consumption by using as an industrial by product that would otherwise go as waste.

ASTM C 618[21] specification breaks fly ash in two classes based on their chemical composition. Class F fly ash has $SiO_2 + Al_2\ O_3 + Fe_2\ O_3$ content of 70% or more and has less than 5% CaO. Class C fly ash has a SiO_2+ Al_2+Fe_2O_3 content between 50% and 70% and more than 29% material is CaO. IS code [17,22] even though does not specify any class of fly ash, the requirement of fly ash to be used in concrete is similar. The coal used in India is predominantly bituminous which give rise to low-lime fly ash similar to Class F. Sub-bituminous lignite coal used in some plants gives high lime fly ash like Class C.

Fly ash varies in colour from light to dark grey depending upon its carbon content, higher the carbon content, darker is its colour. Fly ash is non plastic in character. The specific gravity of fly ash is ranges from 1.90 for sub-bituminous ash to a high value of 2.96 for iron rich bituminous ash. In general, the Blains fineness of Indian fly ash samples varies between 300 and 600m^2/kg except in few stray cases where it is coarser.

Fly ash is constituted of crystalline and amorphous/glassy phases. Both ASTM Class F and Class C fly ashes consist of heterogeneous combinations of amorphous and crystalline phases. The glass phases, generally constituting about 60 to 90 percent of the ash, from when the burned coal residues cool very rapidly and their composition of the pulverized coal and the temperature at which it is burned. The nature of glass present in the fly ash is important and influences significantly its reactivity and consequently its properties as cement making material. Low calcium or Class F fly ashes are characterised by aluminosilicate glass and would show relatively more reactivity in the concrete. Bhanumatidas et al. [23] found that the development of amorphous phase in ASTM type Class F fly ash depends on the basic clay composition in coal and then, operating parameters of power plants such as combustion temperature, fineness and quenching etc.

Incorporation of fly ash in concrete mix improves its properties both in fresh and hardened state. The phenomena, which influence the properties, are principally filler effect and pozzolanic action [24]. The filler effect is immediate, while pozzolanic action occurs later. These two effects results in both pore and grain refinement of the hydrated cementitious system of the mix. The pozzolanic activity of fly ash is greatly influenced by

- The amount and composition of glassy phase present.
- Mineralogical characteristics.
- Particle size of fly ash.

Typically pozzolanic activity of fly ash is proportional to the quantity of particles less than 45 microns size whereas particles larger than this size show a little or no pozzolanic activity. Fly ash when used in concrete, contributes to strength of concrete due to its pozzolanic reactivity. However, since the pozzolanic reaction proceeds slowly, the initial strength of fly ash concrete tends to be lower than that without fly ash.

2.5.3 Ground Ganulated Blast Furnace Slag (GGBFS)

Slag is a waste product in the manufacture of pig iron. Production of every ton of pig iron produces approximately 300 kg of slag. The molten slag is rapidly chilled by quenching in water to form a glassy sand like granulated material [25]. Chemically, slag is a non metallic mixture of lime, silica and alumina that is the same oxides that make up portland cement but not in the same proportions. Blast furnace slag varies greatly in composition and physical structure depending on the process used and on the method used and on the method of cooling of the slag. The granulated slag can be ground to less than 45 micron, will have specific surface of about 400 to 600 m^2/kg, which is finer than portland cement. Increased fineness leads to increased activity at early ages, and occasionally GGBFS with fineness in excess of 500m^2/kg is used. The chemical composition of blast furnace slag is similar to that of cement clinker. The advantages of GGBFS are [26].

- Reduced heat of hydration.
- Refinement of pore structures.
- Reduced permeability to the external agencies.
- Increased resistance to chemical attack.

The presence of GGBFS in the concrete improves the workability and makes the concrete more mobile but cohesive. This is in consequence of a better dispersion of the cementitious particles and of the surface characteristics of the GGBFS particles, which are smooth and absorb little water during mixing. However the workability of the concrete containing GGBFS is more sensitive to variations in the water content of the mix than in the case with portland cement only concrete. When ground to a high fineness, GGBFS reduce bleeding of concrete [25, 27].

The tests on concrete containing GGBFS have confirmed good resistance to penetration by chloride ions. When the content of GGBFS is at least 60 percentages by mass of the cementitious material and the water cement ratio is 0.5, the diffusion coefficient of the concrete exposed to chloride ions is at least ten times smaller than when the cementitious material consists of entirely of portland cement. The very low penetrability of concrete which contains GGBFS is effective also in controlling the alkali silica reaction; the mobility of the alkalis is greatly reduced. This effect is complemented by the incorporation of the alkalis in the products of reaction of GGBFS, especially at the high temperature. The low permeability of well cured concrete containing GGBFS prevents a continuing increase in the depth of carbonation. For this reason, there is no risk of corrosion of steel reinforcement through a reduction in the alkalinity of the hydrated cement paste and depassivation of the steel.

2.6 Self Compacting Concrete (SCC)

For several years, the problem of the durability of concrete structures has been a major problem posed to engineers. To make durable concrete structures, sufficient compaction is required. Compaction for conventional concrete is done by vibrating. Over vibration can easily cause segregation. In conventional concrete, it is difficult to ensure uniform material quality and good density in heavily reinforced locations. If steel is not properly surrounded by concrete, it leads to durability problems. The answer to the problem may be a type of concrete which can get compacted in to every corner of form work and gap between steel, purely by means of its own weight and without the need for compaction. The SCC concept was introduced to overcome these difficulties. This concept can be stated as the concrete that meets special performance and uniformity requirements that cannot always be obtained by using conventional ingredients, normal mixing procedure and curing practices. The advantages of self compacting concrete can be concluded as follows;

- Faster construction.
- Reduction in site man power.

- Better surface finishes.
- Easier placing.
- Improved durability.
- Greater freedom in design.
- Thinner concrete sections.
- Reduced noise levels and absence of vibration.
- Safer working environment.
- Reducing the construction time and labor cost.
- Reducing the noise pollution.
- Improving filling capacity of highly congested structural members.

The SCC is an engineered material consisting of cement, aggregates, water and admixtures with new constituents like colloidal silica, pozzolanic materials like fly ash, ground granulated blast furnace slag (GGBFS), micro silica, metakaolin etc. Chemical admixtures are added to take care of specific requirements, such as high flow ability, compressive strength, high workability, enhanced resistance to chemical and mechanical stresses, lower permeability, durability, resistance against segregation, and passibility under dense reinforcement conditions [28, 29, 30, and 31].

Self compacting concrete is different to ordinary concrete by its ability to fill every kind of form work without any influence from outside. To reach this high performance concrete needs to have special properties. On one hand, SCC should have extremely high flow ability and on the other hand the stability of the paste must be high enough to avoid segregation of coarse aggregates. The properties of hardened SCC should differ as little as possible from those of ordinary vibrated concrete.

In Japan, in early eighties, premature deterioration of concrete structures were detected almost everywhere in the country. The main cause of the deterioration was recognised as inadequate compaction [32]. In addition, the gradual reduction in the number of skilled workers in Japan's construction industry led to a reduction in the quality of construction work. As a solution for these social and technical requirements, the concept of SCC was proposed by Prof. Okamura [33] at Tokyo University in 1988. He gave the first prototype of SCC using materials already in the market. Later studies to develop SCC, including a fundamental study on the workability of concrete, were carried out by Ouchi and Hibino [34].

The European Guidelines [35] for self compacting concrete specification and use is the authentic reference for subject relating to self compacting concrete. These guidelines were prepared by a project group comprising five European Federations dedicated to the promotion of advanced materials and systems for the supply and use of concrete. The self compacting concrete European Project Group was founded in January 2004 and guidelines were recommended in 2005 [35]. European Union guide lines recommend that, the water-binder ratio by volume be 0.8 to 1.10. Total binder content, including powders if any, shall be between 400 and 600 kg/m^3. Water to cement ratio to be selected based on strength and durability requirements. (Water content generally does not exceed 200 l/m^3). Maximum cement content shall be 350 and 450 kg/m^3. Cement having C_3A content more than 10% shall not be used in SCC, because of its role in early setting. It may cause problems of poor workability retention.

Kim et al. cited by Bouzoubaa and Lachemi [36] studied the properties of super flowing concrete containing fly ash and reported that the replacement of cement by 30% fly ash resulted in excellent workability flowability and improve rheological properties.

Zhu and Gibbs [37] in his investigation showed that fine and coarse aggregates could be partially replaced with fly ash in producing high strength self compacting concrete with sufficient flow property and low segregation potential. He further observed that fly ash in SCC helps to improve later age strength beyond 28 days. Ravindrarajah et al. [38] reported similar behaviour when fine and coarse aggregates were partially replaced with fly ash in SCC.

The study on mechanical properties of SCC and the corresponding properties of normal concrete by Persson [39] showed that the elastic modulus, creep and shrinkage of SCC do not differ significantly from the corresponding properties of normal concrete.

Wenzhong et al. [40] studied the uniformity of in-situ properties of self compacting concrete in full scale structural elements. The in-situ concrete properties were assessed by testing cores for in-situ strength, pull-out of pre-embedded inserts and rebound hammer number for near-surface properties. The results indicated that there were no significant differences in uniformity of in-situ properties between the SCC and the corresponding well compacted conventional concrete.

The investigations done by Bouzoubaa and Lachemi [36], on self compacting concrete incorporating high volumes of class F fly ash show that the use of fly ash and blast furnace slag in SCC reduces the dosage of superplasticiser needed to obtain similar slump flow compared to concrete made with portland cement only. Also, the use of fly ash improves rheological properties and reduces cracking of concrete due to the heat of hydration of cement.

Zhu and Peter [41] compared the permeation properties (Permeability, Absorption and Diffusivity) of self compacting concrete with those of traditional vibrated concretes of the same strength. The results indicated that the SCC mixes had significantly lower oxygen permeability and sorptivity than the vibrated normal concretes of the same strength grades. The study also concluded that the chloride diffusivity appeared to be dependent on the type of filler used.

Sekhar and Rao [42] conducted tests on SCC and proposed the following mathematical equations (2.3 to 2.4) relating compressive strength, split tensile strength and flexural strength of SCC.

$$f_s = 0.08 f_{ck}^{\ 1.04} \qquad (2.3)$$

$$f_t = 0.12\, f_{ck}^{\ 0.9} \qquad (2.4)$$

Mohammed Abdul Hameed [43] conducted a study of mix design and durability of self compacting concrete and reported that the compressive strength of SCC increased with the time of curing. Considerable increase in the compressive strength of concrete specimens exposed to thermal variations was observed compared to specimens exposed to normal exposure.

Gettu et al. [44] have evaluated about robustness of SCC and found that the increase in cement content leads to a low slump flow but decrease in cement content decreases the compressive strength of concrete. Changes in the fly ash dosage do not significantly alter the self compatibility. Variation of super plasticiser affects the flowability of the mix but it does not influence the strength and observed that self compacting concrete is generally less tolerant to changes in the characteristics and dosages of its constituents.

Parra et al. [45] studied splitting tensile strength and modulus of elasticity of self compacting concrete and reported that, tensile strength is related to compressive strength and in general tensile strength is lower in SCC than in normal vibrated concrete. This is due to the factors that affect aggregate-paste bond, which thus have a greater influence on tensile than compressive strength.

Gettu et al., cited by Parra et al. [45] suggest that the formation of large calcium hydroxide crystals and ettringite with the usage of polycarboxylate type superplasticisers in SCC weakens the aggregate-paste transition zone and, as a result, decreases the concrete's tensile strength.

SCC has now been taken up with enthusiasm across Europe and other parts of the world, in both site and precast concrete work. Practical application has been backed up by research on its physical and mechanical characteristics of SCC.

In the 1980's Canada Centre for Mineral and Energy Technology (CANMET) designed the so called high volume fly ash (HVFA) concrete. In this concrete 55-60% of the portland cement was replaced by Class F fly ash and this concrete demonstrated excellent mechanical and durability properties [46].

Khatib [47] found in a study on performance of self compacting concrete containing fly ash, that high percentage of FA can be used to produce SCC with an adequate strength. He advised 40MPa strength for SCC with 60% replacement with fly ash. He further observed that increasing amounts of FA in SCC reduces the drying shrinkage and replacing cement with 80% FA can reduce the shrinkage by two third.

Sukumar et al. [48] conducted study on evaluation of strength at early ages of self compacting concrete with high volume fly ash and proposed expression for the strength development of SCC with age, which is in line with that of BIS relation for normal concrete [49].

$f_{ct} = f_{ck} \times t/(4.2+0.85 \times t)$ .... (2.5)

A relation between split tensile strength and compressive strength of SCC has also been proposed by them, which is given in equation (2.6).

$f_s = 0.0843 f_{ck} + 0.818$ ....(2.6)

Compressive strength of SCC at various ages of curing are found to be comparable with the relation suggested by Bureau of Indian standards [49].

Mattur et al. [50] studied about the strength of high volume fly ash SCC and found that fly ash based binary blends show good flow and strength properties as required for SCC, with use of superplasticiser, but with extended setting times, which may create problems associated with removal of formwork. He further reported that use of fly ash or GGBFS based ternary blends improve the setting characteristics of the SCC, but do not achieve the required flow properties of SCC.

2.7 Concrete Exposed to High Temperature

It is important that buildings and structures are made in such a way that risk to both people and property are minimised as effectively and efficiently as possible. In India, most of the industrial buildings are made up of steel structures. Fire remains one of the most serious potential risks to buildings especially for industrial structures made with steel. Even though there exists many methods to protect steel structures from fire, encasing it with non conducting materials (Jacketing) is the preferred one. One of such material for fire protection is concrete, which is a non-combustible material and has a low rate of thermal conductivity. Buchanan cited by Li and Purkiss [51] reported that concrete cannot be 'set on fire' unlike other materials in a building and does not emit any toxic fumes when affected by fire. It will also not produce smoke or drip molten particles, unlike some plastics and metals, so it does not add to the fire load.

With the fast depleting state of natural resources like sand and aggregate, it is time to look for alternative materials for making concrete. Since performance is more important than strength when exposed to fire, concrete made using marginal materials could be seriously considered for protecting steel against fire. When concrete is exposed to high temperatures, changes in mechanical properties and degradation in durability occurs. Nonlinearities in material properties, variation of mechanical and physical properties with temperature, tensile cracking and creep effects affect the buildup of thermal forces; the load carrying capacity and ductility of the structural members are also affected. The property variations result largely because of changes in the moisture condition of the concrete constituents and the progressive deterioration of the cement paste aggregate bond, which is especially critical where thermal expansion values for the cement paste and aggregate differ considerably. When concrete made with pozzolanic cement is subjected to heat, a number of transformations and reactions take place even when only a moderate increase in temperature occurs.

Since aggregate occupy 65 to 75% of the concrete volume, the behavior of concrete at elevated temperature is strongly influenced by aggregate type. Commonly used aggregate materials are thermally stable up to 300-350°C [52]. Aggregate characteristics that influence the behavior of concrete at elevated temperature include thermal conductivity, thermal expansion, chemical stability and integrity. Some siliceous or calcareous aggregates with some water of constitution exhibit moderate de hydration with increasing temperature that is accompanied by shrinkage (for example Opel at 373°C exhibits shrinkage of 13% by volume). Most non-siliceous aggregate are stable up to about 600°C. At higher temperatures, calcareous aggregates dissociate into an oxide. Calcium carbonate dissociates completely at 898°C under atmospheric pressure. Above 1200°Cand up to 1300°C some aggregates, such as igneous rocks show degassing and expansion. It has been noted that the thermal stability of aggregates increases in order of gravel, limestone and basalt [53].

Apart from the crystalline transformations occurring mainly in the aggregate materials during heating, a number of degradation reactions occur, primarily in the cement paste, that result in a progressive breakdown in the structure of concrete. An increase in temperature produces changes in the chemical composition and microstructure of hardened portland cement paste. At ambient temperature, evaporable water occupies about 30 to 60% of the volume of saturated cement paste. As the temperature to which the cement paste is subjected to increase, evaporable water is driven off until a temperature of about 105°C, all evaporable water will be lost, given a sufficient exposure period. At temperatures above 105°C, the strongly absorbed and chemically combined water (water of hydration) are gradually lost from cement paste hydrates with the dehydration essentially complete at 850°C. Dehydration of the calcium hydroxide is essentially zero up to about 400°C, increases most rapidly around 535°C, and becomes complete at about 600°C [54].

A neat portland cement paste on heating first expands owing to the normal thermal expansion; this expansion is opposed, however by a contraction due to the shrinkage of the material as water is driven off from it. The contraction due to drying eventually becomes much larger than the normal thermal expansion and the material then commences to shrink. The actual temperature at which the maximum expansion is reached varies with the size of the specimen and the condition of heating. It may be as high as 300°C for air dry specimens under conditions of fairly rapid heating. At higher temperatures the neat cement paste steadily shrinks, the contraction from the original dimensions amounting ultimately to 0.5%, or more. During this process cracking occur in cement paste. The effect of increase in temperature on strength of concrete is not much up to temperature of about 250°C, but above 300°C, definite loss of strength take place. The loss of strength may be about 50% or more at about 500°C [55].

The rate of increase of temperature through the cross section of a concrete element is relatively slow. This means that the internal zones of the concrete do not reach the same high temperatures as a concrete surface exposed to flame.

Concrete is subject to a form of damage due to excessive heat which is known as spalling. This takes the form of the breaking off pieces or layers of the material, sometimes with explosive violence. Spalling is caused by one or more of the following causes.

- Excessive compression or restraint of the material.
- The formation of high pressure stem in the material.
- Splitting of the aggregate used in the concrete mix.

The aggregates present in mortar and concrete undergo a progressive expansion on heating, while the hydrated products of the set cement, beyond the point of maximum expansion, shrinks. These two opposing actions progressively weaken and crack the concrete. The various aggregates used differ considerably in their behavior on heating. Quartz, the principle mineral in sand, granites and gravels expands steadily up to about 573°C. At this temperature it undergoes a sudden expansion of 0.85%, caused by the transformation of low quartz to high quartz. This expansion has a disruptive action on the stability of concrete. The fire resisting properties of concrete is least, if quartz is the predominant mineral in the aggregate. The best fire resistant aggregates amongst the igneous rocks are basalts and dolomite. Limestone expands steadily until a temperature of about 900°C and then begins to contract owing to decomposition with liberation of carbon dioxide. Since the decomposition takes place only at a very high temperature of 900°C, it has been found that dense limestone is considered as a good fire resistant aggregate. Perhaps the best fire resistant aggregate is blast furnace slag aggregate. Broken bricks also form a good aggregate in respect of fire resistance. The long series of tests indicated that even the best fire resistant concrete have been found to fail if concrete is exposed for a considerable period to a temperature exceeding 900°C, while serious reduction in strength occurs at a temperature of about 600°C [56]. If held permanently exposed to heat, any cement product must suffer considerable loss in strength and undergo gradual breakdown at temperatures which are considerably lower than 600°C. It is found that mortars heated for 10 hours at 300°C suffered little loss in strength, but that at 500°C there was a loss which increased with time up to 6 hours, when about half of the original strength has been lost [57].

Butcher and Pranell [58] reported that, the behaviour of concrete when heated will depend on the aggregate used, on the moisture content, on the cement to aggregate ratio, and on the applied load. They reported that, up to temperatures of 200°C, only a slight reduction occurs in concrete strength but at a temperature of 500°C and above strength of concrete deteriorate rapidly.

Roytman [59] reported that the strength of most structural materials increases slightly in the range of temperatures of the order of 200 to 300°C and then tends to decrease with further rise in temperature.

Poon et al. [60] studied performance of metakaolin concrete at elevated temperatures and pointed out that the addition of silica fumes highly densifies the pore structure of concrete, which can result in explosive spalling due to the build-up of pore pressure by steam. On the other hand, the addition of fly ash or GGBFS enhances the fire resistance of concrete. Moreover, the fly ash concrete retained higher strength than the pure OPC concrete at higher temperatures up to 650°C and completely eliminated all visible surface cracks for specimens heated up to 600°C. The GGBFS concrete showed the best performance followed by fly ash and silica fumes concretes.

Youssef et al. [61] in the paper general stress strain relationship for concrete at elevated temperature discussed about various models to predict the compressive strength and modulus of elasticity of concrete at elevated temperature from the ambient values.

Lie cited by Youssef and Moftah [61] proposes prediction equation in 3 temperature zones. Which are reproduced equations (2.7a to 2.7c);

$f_{cT}=f_{ck}(1-0.001 \,.\, T) \quad T \leq 500°C$ ....(2.7a)

$f_{cT}=f_{ck}(1.375-0.001175 \,.\, T) \quad 500°C \leq T \leq 700°C$ ....(2.7b)

$f_{cT}= 0 \quad T \geq 700°C$ .... (2.7c)

The Eurocode cited by Youssef and Moftah [61] also predicts the strength of concrete with temperature in three temperature zones, and the same is given as equations (2.8a to 2.8c).

$f_{cT} = f_{ck} \quad T \leq 100°C$ .... (2.8a)

$f_{cT} = f_{ck}(1.067 - 0.00067 . T)\ 100°C \leq T \leq 400°C$ .... (2.8b)

$f_{cT} = f_{ck}(1.44 - 0.0016 . T) \geq 0 T \geq 400°C$ ....(2.8c)

The Lie and Lin model cited by Youssef and Moftah [61] proposes a single equation to predict the strength of concrete with temperature and is shown in equation (2.9).

$f_{cT} = f_{ck}(2.011 - 2.353(T-20)/1000) \leq f_{ck}$ ....(2.9)

Li and Purkiss [51] proposed a third order equation to predict the strength variation in concrete with temperature, which is given in equation (2.10).

$f_{cT} = f_{ck}[0.00165(T/100)^3 - 0.03(T/100)^2 + 0.025(T/100) + 1.002]$ .....(2.10)

Hertz cited by Youssef and Moftah [61] proposed a model that recognizes the influence of type of aggregates on the strength variation of concrete with temperature and the same is given in equation (2.11).

$f_{cT} = f_{ck}[1/\{1+(T/T_t)+(T/T_2)^2+(T/T_8)^8+(T/T_{64})^{64}\}]$ ....(2.11)

In the above equation, values for T_t, T_2, T_8 and T_{64} are different for siliceous, light weight and other types of aggregates.

Li and Guo cited by Xiao and Konig [62] in the paper study on concrete at high temperature in China proposed a model for the prediction of strength variation with temperature as given by equation (2.12).

$f_{cT} = f_{ck}/[1+2.4(T-200)^6 \times 10^{-17}]$ .... (2.12)

Researchers have proposed models for tensile strength of concrete under high temperature. An accurate prediction of tensile strength of concrete will help in mitigating cracking problems, improve shear strength prediction and minimise the failure of concrete in tension due to inadequate methods of tensile strength prediction.

Li and Guo cited by Xiao and Konig [62] suggested a simplified equation as given by equation (2.13) for the prediction of tensile strength of concrete (f_{TT}) with temperature up to 1000°C.

f_{TT} = (1-0.01T)f_T (20-1000°C) .... (2.13)

Xie and Qian cited by Youssef and Moftah [61] proposed a second- order fitting formula and a simplified two-part formula to estimate the tensile strength of concrete as given by equation (2.14), (2.15a and 2.15b) respectively.

f_{TT} = [2.08(T/100)2 – 2.666(T/10) +104.79] f_T (20-1000°C) .... (2.14)

f_{TT} = [0.58(1.0 – T/300) + 0.42] f_T (20-300°C) .... (2.15a)

f_{TT} = [0.42 (1.6- T/300) +0.42] f_T (300-800°C) .... (2.15b)

Francis cited by Sukumar et al. [48], reported a relation between split tensile strength and cylinder compressive strength (f_{cy}) of concrete as in equation (2.16).

$f_T = 0.206\ (f_{cy})^{0.79}$ .... (2.16)

Prediction models are available for elastic modulus of concrete also. Xiao and Konig [62] cited that Lu suggested a tri-linear model expression between E_T and T which are given in equation (2.17a) to (2.17c).

E_T = (1-0.0015T)E_C (20-300°C) .... (2.17a)

E_T = (0.87-0.0084T)E_C (200-700°C) .... (2.17b)

E_T = 0.28 E_C (>700°C).. (2.17c)

Li and Guo cited by Youssef and Moftah [61] and Xiao and Konig [62] suggested a bi-linear equation between E_T and T, which is represented as follows.

$E_T = E_C$ (20-60°C).. (2.18a)

$E_T = (0.83-0.0011T)\ E_C$ (60-700°C) ....(2.18b)

Khennane and Baker cited by Youssef and Moftah [61] proposed a model for unloaded concrete and are reported by equation (2.19).

$E_T = (-0.001282\ T + 1.025641)\ E_C \quad 20 \leq T \leq 800°C$ ....(2.19)

2.8 Fire Resistance of Self Compacting Concrete

Fire resistance is a property of materials that prevents or retards the passage of excessive heat or flames under conditions of use. For building elements, it is defined as the ability of an element to continue its structural function without affecting its structural capacity, integrity and insulation property for a stated period of time. The main reasons for failure of a concrete element at high temperatures are spalling and loss of strength. Moreover, the compressive strength of concrete has an important influence on its fire behaviour. A higher compressive strength is usually seen with more packing and less porosity, which may lead to higher pore pressure and spalling. The fire behaviour of new concretes, like the SCC and high performance concrete (HPC) is very different compared with traditional concretes. The kind of powder in SCC may significantly affect both of this behaviour.

Sideris [63] studied the mechanical characteristics of self consolidating concretes exposed to elevated temperatures and found that the residual compressive strength of SCC mixtures was higher than the one of conventional concrete for the same strength class. He also reported that explosive spalling occurred in both SCC and conventional concrete of the highest strength category at temperatures greater than 380ºC. He also observed that, the influence of elevated temperatures was more detrimental to splitting tensile strength.

Tayfun and Topcu [64] observed that the compressive strength of SCC decreased by increasing the water-powder ratio of concrete.

Fares et al. [65,66] examined the mechanical properties (compressive strength, flexural strength and modulus of elasticity) of self compacting concrete subjected to elevated temperature and found that they are lower than the vibrated concrete. The risk of spalling is more pronounced for SCC than for vibrated concrete.

Ali [67] conducted investigation on residual mechanical properties of self compacting concrete with high reactivity metakaoline exposed to elevated temperature and found that at the same exposure temperature the rate of loss in modulus of rupture is higher than that of compressive strength.

Siddique and Kaur [27] investigated the properties of concrete containing ground granulated blast furnace slag at elevated temperature and reported that at temperatures between 200 and 350ºC, the mass loss is not very significant and there was no very significant deterioration of the mechanical properties of concrete between 27 and 100ºC.

Bakhtiyari et al. [68] reported that SCC is more susceptible to spalling relative to normal vibrated concrete (NC). The tensile strength is decreased faster than the compressive strength at higher temperatures, due to the characteristics of the transition zone between the paste and the aggregate.

Earlier researchers investigated the effect of high temperature on mechanical properties of self-compacting concrete.

2.9 Fire Resistance of Laterised Concrete

Balogun [69] examined the effect of temperature on the residual compressive strength of laterised concrete. He observed that the gain in strength depends on sand content and density of laterised concrete was not significantly affected by changes in temperature.

Brooks [70] conducted an experimental program to investigate the strength performance of laterised concrete subjected elevated temperatures of 200, 400 and 600ºC, incorporating 0, 10,20,30,40 and 50% laterite aggregate as a replacement by weight of sand. He found that laterised concrete experienced strength losses that increased with temperature. This study further reveals that air cooled laterised concrete maintained higher residual strength values than water cooled concrete. He has recommended that the Euro code and CEB design curve could be applied to laterised concrete subjected to temperature below400ºC.

Ikponmwosa and Salau [71] conducted studies on effect of heat on laterised concrete and found that the compressive strength of normal concrete increases with temperature up to 250ºC. However, concrete with 25% laterite as fine aggregate can withstand heat up to 500ºC. There should be a considerable economic saving if laterised concrete is used in areas of high temperature up to 500ºC. In such situation the application of abundant and locally available laterite is ideal as a construction material.

Study by Udoeyo et al. [72] on residual compressive strength of laterised concrete subjected to elevated temperatures, observed that, the compressive strength of laterised concrete decreases in a similar manner to that of plain concrete when subjected to elevated temperatures between 200ºC and 600ºC.

Review of literature shows that the thermal properties of laterised concrete addressed only compressive strength and on laterised concrete obtained by replacing fine aggregate (sand) by laterite fine aggregate. No study has been conducted on self compacting concrete with laterite aggregate. Hence detailed studies are needed to understand the behaviour of laterised concrete subjected to elevated temperature.

2.10 Shear Strength of Concrete

Shear strength is an area of major importance for all concrete materials and many concrete structures. Despite the decades of research, less information is available with regards to the shear strength of plain laterised concrete and current design is still based on empirical results. Most of the research effort on the application of fracture mechanics to concrete materials has concentrated on Mode I failure. Mode II failure has received very little attention due to the fact that it is not an easy mode to test. Hence indirect methods like Brazilian test, Double Punch Test (DPT), 3 and 4 point bending test are often preferred to direct uniaxial tests. However, shear failure is an area of major importance in concrete structures.

Bazant and Pfeiffer [73] conducted tests on symmetrically notched beam specimens of concrete and mortar, loaded near the notches by concentrated forces that produce a concentrated shear force zone, are tested to failure. Thus, the failure is due essentially to shear fracture (Mode II). The crack propagation direction seems to be governed by maximum energy release rate. They found that, like the tensile (Mode I) fracture, the shear (Mode II) fracture follows the size effect law. The shear (Mode II) fracture energy for the type of test adopted appears to be about 25 times larger than the tensile (Mode I) fracture energy.

Barr [74] proposed a compact shear test specimen for Mode II shear fracture test. The main advantage of the test geometry proposed by him is that, it may be prepared from either standard cylinders or from concrete cores. He calculated Mode II stress intensity factor based on the method proposed by Bazant and Pfeiffer [73].

Bazant and Prat [75] measured fracture energy under mode III (Torsion) failure and found that, this value is about three times larger than Mode I fracture energy and about 9 times smaller than the Mode II fracture energy.

Salau and Balogun [76] investigated shear resistance and deflection of reinforced laterised concrete beams without shear reinforcement. The concrete containing laterite at 25%, 50%, 75% and 100% of total fine aggregate content were considered for the study. They found that the mode of failure does not depend on the percentage laterite content but depends only on the shear span and the ultimate cracking load decreases with increase in the percentage laterite content. The results further show that the presence of laterite in the concrete improves its post-cracking ability and serviceability conditions due to high ductility, stiffness and superior crack control of lateritic content in comparison with plain concrete.

Basche et al. [77] analysed numerically the shear capacity of lightweight concrete beams. They observed large rotation due to shear failure.

Pros et al. [78] numerically modeled the double punch test for plain concrete. They found that the analytical solution is in agreement with indirect tension tests (Brazilian test and the double punch test)

It may be noted that limited experimental study has been reported for the mode II type fracture test on concrete. No result is available for the laterised self compacting concrete.

2.11 Fire Resistance of Ferrocement

Fire remains one of the most serious potential risks to most building and structures. Most structural materials which are weakened when exposed to high temperatures, cause buildings to collapse. Therefore the use of fire protection materials to reduce thermal damage of structural members is important and necessary. Ferrocement is one of the cementitious composite materials used for fire protection to shield structural element. The integrity and insulation performance are very important properties for fire protection materials like ferrocement. The mechanical properties of ferrocement by itself are not very good when exposed to fire, because it is a thin material with a small cover to the reinforcement. However the use of jacketed ferrocement with other structural components like reinforced concrete, prestressed concrete or steel may enhance the fire resistance of composite element. For reinforced concrete or prestressed concrete structures, even though concrete covering can be a good insulator to resist heat transmission from fire to reinforcing steel bar or prestressing steel wire, the spalling of concrete is a problem when exposed to fire, especially for high strength concrete; it may cause collapse of the reinforced concrete or prestressed concrete structures. Since mortar is a good insulator as well as concrete and the reinforcing mesh can reduce the spalling of the ferrocement matrix and probably prevent the collapse of the concrete structures when it is exposed to fire. The study on the behavior of ferrocement under fire is still in its infancy.

Abdullah et al. [79] investigated the thermal behavior of ferrocement subjected to Missile impact. They found that increasing volume fraction of reinforcement does not improve impact resistance significantly.

Greepala and Nimityongskul [80] investigated the structural integrity and insulation performance of ferrocement exposed to fire. The parameters investigated include thickness of ferrocement, mortar covering and specific surface of wire mesh. To meet the integrity criterion, the test specimen must not develop any cracks or fissures which allow smoke or hot gases to pass through the assembly. The test results revealed that ferrocement specimens met the structural integrity criterion in accordance with ASTM standard. An increase in the thickness of ferrocement, from 15mm to 25mm, significantly increased the insulation performance of ferrocement during the first 75 minutes of fire exposure. Subsequently, the increase in thickness had less influence on the insulation performance of ferrocement. For the same thickness of ferrocement, it was found that the increase in specific surface of wire mesh resulting in a poorer in insulation property of material.

The mechanical properties of ferrocement jacket were experimentally determined based on its flexural characteristics after exposure to fire by Greepala and Nimityongskul [81]. Test results showed that ferrocement jacket is a satisfactory solution for fire protection due to its post-fire strength as compared with those of plain mortar. They observed that, an increase in wire mesh content significantly improved the mechanical properties of ferrocement under normal condition; however after fire exposure, the content of wire mesh was no longer significant regardless of heating duration. Mortar covers had negligible influence on mechanical properties of ferrocement jacket exposed to fire for both short and long duration of heating.

Greepala and Nimityongskul [82] experimentally investigated the structural integrity of ferrocement jacket exposed to fire. Structural integrity is obtained experimentally from the flexural characteristics and damage to ferrocement panels after exposure to fire. The investigated parameters are the volume fraction of mesh and mortar cover. Tests showed that before exposure to fire, it was noted that an increase in wire mesh content significantly increased the flexural strength and toughness of ferrocement. However the post-fire flexural strength and toughness of ferrocement barely dependent on the content of wire mesh. Thus, it can be seen that using high wire mesh content is not the right solution to improve the post-fire flexural strength and toughness of ferrocement jacket, in other words the post–fire mechanical properties of the ferrocement jacket only slightly increased and a higher volume fraction of wire mesh induces delamination of mortar and results in in-plane cracking. The mortar cover had no significant influence on the flexural strength and toughness of the ferrocement jacket both before and after exposure to fire. It is also observed that the initial cracks correspond to skeletal steel.

Even though limited study has been done on the behavior of ferrocement elements under elevated temperature, no study has been reported in the case of ferrocement elements with laterised concrete and laterised self compacting concrete.

2.12 Concluding Remarks

From the review of available literature it could be concluded that;

- Large laterite deposits are available in many countries. Laterite terrain of Kerala covers about 60% of its area.
- Even though use of laterite as a building blocks dates back to the early civilisation, it is under utilised for other applications.
- Even though laterite deposit is soft, with weathering action, it forms as a loamy soil with iron oxide pellets and gravels.

- The past researchers confined their study by replacing sand with laterite fine aggregate in concrete. To the best of the author's knowledge, no attempt has been reported about the judicious use of laterite coarse aggregate as well as all-in aggregate in concrete.
- Suitable supplementary cementitious materials, when used in appropriate quantity in concrete modify certain properties of fresh and hardened concrete.
- Past study on thermal properties of laterised concrete addressed only the compressive strength that too on laterised concrete obtained by replacing fine aggregate (sand) by laterite fine aggregate.
- Even though study on self compacting concrete is gaining momentum, no study has been reported on self compacting concrete with laterite aggregate.
- Only limited experimental study has been reported for the mode II type fracture test on concrete in general and no study has been reported for concrete with laterite aggregate.
- No study has been reported about the behaviour of ferrocement elements with laterised concrete as well as on laterised self compacting concrete.
- Concrete is one of the fire protection materials for steel structural members. Behaviour of concrete when exposed to high temperature has been studied in detail in the past.
- Self compacting concrete has the potential to be used in jacketing of structural members due to its established advantages like flowability and fillability.

- Use of jacketed ferrocement with other structural components enhances the fire resistance of the composite elements.
- To the best of the author's knowledge, limited attempt has been made to study the behaviour of concrete made with marginal materials at high temperatures for the purpose of using it as a fire protection material. Laterite aggregate is one of such potential marginal materials to be used in concrete.

2.13 Objectives

Based on the literature survey and having identified the gap areas, the major objectives of the present investigation are outlined as follows.

- To study the feasibility of using weathered laterite aggregate in concrete.
- To propose a mix design methodology for laterised self compacting concrete.
- To study the thermal behaviour of laterised concrete as well as laterised self compacting concrete and to check its viability as a fire protection material.

2.14 Scope

In order to achieve the above stated objectives, the scope of present research has been limited to the following.

- To decide one specific grade of normal strength concrete for present study.
- To design mix proportion for the grade of concrete selected using conventional aggregates.

- To arrive at a mix proportion for the corresponding laterised concrete by replacing fine aggregate and coarse aggregate of conventional concrete with weathered laterite aggregates.
- To determine the physical and mechanical properties of laterised concrete by varying the following parameters;

  (a) Source of laterite aggregate

  (b) Grades of ordinary portland cement

  (c) Supplementary cementitious materials (Fly ash and GGBFS)
- To compare the test results of laterised concrete and corresponding conventional concrete and to arrive at specific conclusions.
- To propose a mix design methodology for normal strength laterised self compacting concrete based on trial mixes and in line with modified Nan Su method.
- To validate the proposed mix design methodology by testing the physical and mechanical properties of laterised self compacting concrete.
- To compare the physical and mechanical properties of different types of concrete after exposure to various temperature levels and cooled subsequently under two different methods (air cooled and water cooled). The types of concrete to be considered include;

  (a) Conventional concrete with ordinary portland cement and supplementary cementitious materials.

  (b) Laterised concrete with ordinary portland cement and supplementary cementitious materials.

(c) Laterised self compacting concrete with different types of additions.

(d) Ferrocement elements with laterised self compacting concrete and with different cover to mesh reinforcement.

- To fabricate an experimental setup to conduct mode-II fracture test on concrete specimen and to carry out a preliminary study on shear strength of laterised self compacting concrete.

Chapter-3

MATERIALS AND METHODS

3.1 Introduction

This chapter deals with the details about various materials used for the present study and their test results. The method of testing of self computing concrete, heating of specimen etc. is also mentioned briefly in this chapter. Detailed methods for testing of self compacting concrete are presented in appendix-1 for ready reference.

An experimental setup for the determination of shear strength (Mode II fracture) of concrete has been fabricated and its details along with test procedure have been presented in this chapter.

3.2 Materials

The common ingredients of concrete are cement, coarse and fine aggregates and water. A fourth ingredient called admixture is used to modify certain specific properties of concrete in fresh and hardened stage. The physical and chemical properties of each ingredient has considerable role in the desirable properties of concrete like durability, strength, and workability.

3.2.1 Cement

Ordinary Portland Cement (OPC) of Grade 33, 43 and 53, has been used in the present study. The cement considered has been tested as per relevant Indian standard code of practices [83]. Table 3.1 present the results of the tests conducted on cement. The results have been compared with standard values

[84, 85, 86, 87 and 88] and found that all the test results comply with the respective specifications.

Table 3.1 Physical properties of cement

Sl. No.	Properties	OPC 33 Grade	OPC 43 Grade	OPC 53 Grade
1	Specific gravity	3.11	3.12	3.14
2	Standard consistency (%)	30.60	31.00	33.60
3	Initial setting time (min.)	89	100	80
4	Final setting time (min.)	260	252	240
5	Soundness by Le- Chatelier (mm)	3	4	3
6	Average 28th day cube compressive strength (MPa)	36.20	54.00	61.40

3.2.2 Fine Aggregate

Fine aggregate (Sand) collected from Periyar and weathered laterite collected from Cochin has been used for the present investigation. The material has been tested as per Indian Standard Code of Practices [89] and results have been found satisfying the relevant Indian Standard Specifications [90]. The physical properties of fine aggregate are presented in Table 3.2.

3.2.3 Coarse Aggregate

Coarse aggregate is considered to be the strongest and least porous component of concrete. It should be also a chemically stable material. The strength of concrete cannot exceed that of the bulk of aggregate contained therein. Locally available crushed granite aggregate of 20mm down size conforming to Indian Standard Specification [90] was used as coarse aggregate for the present work. The physical

Table 3.2 Physical properties of fine aggregate

a) Particle size distribution			
Sl. No.	**Sieve size(mm)**	**Percentage finer**	
		Sand	**Weathered laterite**
1	12.50	100	100
2	10.00	100	100
3	4.75	98.9	98.8
4	2.36	93.4	92.9
5	1.18	69.9	72.8
6	0.60	40.3	42.9
7	0.30	12.8	12.0
8	0.15	2.3	2.2

b) Other properties			
Sl. No.	**Properties**	**Sand**	**Weathered laterite**
1	Grading	Zone II	Zone II
2	Fineness modulus	2.84	2.78
3	Uniformity Coefficient	3.41	3.16
4	Specific gravity	2.67	2.52
5	Bulking	33.3%	38.2%
6	Water absorption	0.9%	12.0%

properties of coarse aggregate are tabulated in Table 3.3. Figure 3.1 depicts the plot of particle size distribution curve for fine aggregates. The particle size distribution of laterite fine aggregate has been made close to that of river sand by properly grading the former.

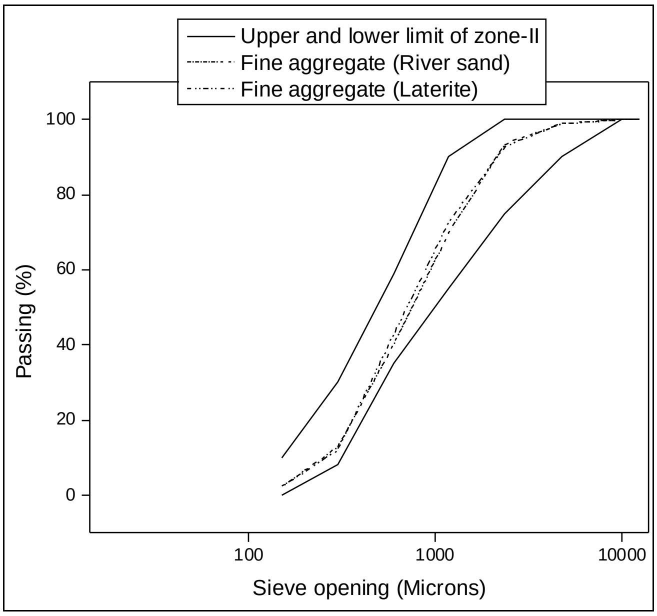

Figure 3.1 Particle size distribution curve for fine aggregates

3.2.4 Weathered Laterite All-in Aggregate

For the present study, 12mm nominal size weathered laterite all-in aggregate has been used. Table 3.4 compares the physical and chemical properties of weathered laterite all-in aggregate collected from a wide range of sources like Kollam, Cochin, Malappuram and Kasaragod.

Figure 3.2 depicts the plot of particle size distribution curve for laterite aggregate collected from various sources like Kollam, Cochin (CUSAT), Malappuram and Kasaragod.

From Figure 3.2, it has been observed that the weathered laterite aggregate, collected from all sources satisfy the required specification for all-in aggregate as per Indian Standard Specification [90]. The chemical composition of laterite samples collected has been determined at Centre for Earth Science and Studies (CESS), Thiruvananthapuram and the results are presented in Table 3.5. Typical photograph of weathered laterite are shown in Figures 3.3 and 3.4.

Table 3.3 Physical properties of coarse aggregate (Crushed granite)

a) Particle size distribution		
Sl. No.	**Sieve size (mm)**	**% of finer**
1	20.00	97.50
2	12.50	50.00
3	10.00	38.50
4	4.75	0.0
5	2.36	0.0
6	1.18	0.0
7	0.60	0.0
8	0.30	0.0
9	0.15	0.0
b) Other physical properties of coarse aggregate		
Sl. No.	**Properties**	**Value**
1	Fineness modulus	7.56
2	Aggregate crushing value	26.00%
3	Specific gravity	2.77
4	Water absorption	0.20%

3.2.5 Water

The amount of water should theoretically be enough for complete hydration of cement and should not contain any harmful materials in it. Potable water has been used for making concrete in the present study.

Table 3.4 Physical properties of weathered laterite all-in aggregate from various sources.

a) Particle size distribution

Sl. No.	Sieve size (mm)	% of finer			
		Kollam	Cochin (CUSAT)	Malappuram	Kasaragod
1	20.00	100.00	100.00	100.00	100.00
2	12.50	92.50	100.00	97.80	87.10
3	10.00	80.00	86.30	71.90	78.50
4	4.75	43.20	44.70	42.90	39.70
5	2.36	33.40	19.80	37.00	28.70
6	1.18	24.60	13.30	26.60	20.90
7	0.60	13.30	10.70	17.10	11.00
8	0.30	3.80	9.00	12.20	2.70
9	0.15	1.30	5.60	2.50	0.60

b) Other physical properties

Sl. No.	Properties	Value			
		Kollam	Cochin (CUSAT)	Malappuram	Kasaragod
1	Fineness modulus	6.09	7.00	6.92	6.30
2	Aggregate crushing value (%)	29.88	29.00	29.80	29.90
3	Specific gravity	2.68	2.71	2.62	2.66
4	Bulk density (g/cc)	1.62	1.65	1.58	1.62
5	Water absorption (%)	10.00	12.00	11.00	12.00

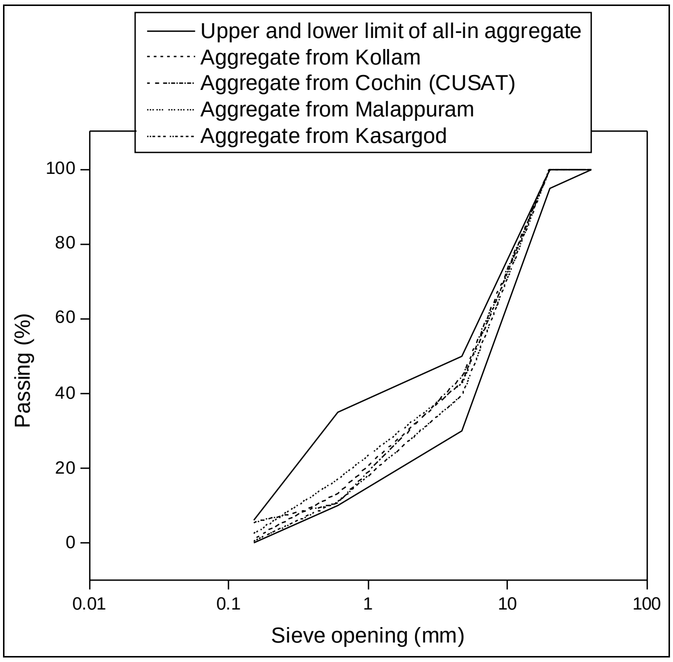

Figure 3.2 Particle size distribution curve for laterite coarse aggregates

Figure 3.3 View of weathered laterite aggregate deposit from Cochin (CUSAT)

Table 3.5 Chemical properties of weathered laterite all-in aggregate collected from various sources.

Sl. No.	Compound	Values (%)			
		Cochin (CUSAT)	Kollam	Malappuram	Kasaragod
1	SiO_2	30.950	21.630	24.320	15.220
2	TiO_2	0.686	1.070	0.980	1.450
3	Al_2O_3	21.520	23.540	24.320	26.860
4	MnO	0.018	Not Detected	0.012	Not Detected
5	Fe_2O_3	35.800	41.840	39.230	42.850
6	CaO	0.123	0.080	0.092	0.064
7	MgO	Not Detected	Not Detected	Not Detected	Not Detected
8	Na_2O	Not Detected	Not Detected	Not Detected	Not Detected
9	K_2O	0.084	0.046	0.048	0.060
10	P_2O_5	0.198	0.173	0.182	0.289
11	Loss on Ignition	10.220	11.120	10.320	12.570

3.2.6 Superplasticisers

Requirement of right workability is the essence of good concrete. Concrete in different situations require different degree of workability. Organic substances or combination of organic and inorganic substances, which allow a reduction in water content for the given workability or give a higher workability at the same water content, are termed as plasticising admixtures. Superplasticiser constitute a relatively new category and improved version of plasticiser. The use of superplasticisers is

Figure 3.4 Closer view of weathered laterite aggregate deposit from Cochin (CUSAT)

practiced for production of flowing, self leveling, self compacting concrete and for the production of high strength and high performance concrete. Two types of Superplasticiser have been used in the present study, namely Rheobuild SP-1i and Glenium B-233.

Superplasticiser used in the present investigation for preparing laterised concrete was commercially available high range water reducing, set retarding, Superplasticiser known as Rheobuild SP-1i. A rheoplastic concrete is a fluid concrete with a slump of at least 200mm easily flowing but at the same time free from segregation and having the same water cement ratio as that of a no slump concrete (25 mm) with admixture.

Rheobuild SP-1i is a Sulphonated Naphthalene Formaldehyde (SNF) based synthetic polymer specially designed to impart rheoplastic qualities to concrete, manufactured by BASF construction Chemicals Private Ltd. The physical and chemical properties of Rheobuild SP-1i as per the manufacturer are listed in Table 3.6 and satisfy Indian Standard Specifications [91].

Table 3.6 Typical properties of Rheobuild SP-1i Superplasticiser

Sl. No.	Particulars	Value/Description (as per manufacturer)
1	Aspect	Dark brown free flowing liquid
2	Relative Density	1.18 ± 0.02 at 25°C
3	pH	≥ 6
4	Chloride ion content	$<$ 0.2%
5	Dosage	600 *ml* per 100 *kg* of cement

Superplasticiser or high range water reducing admixture is an essential component of self compacting concrete. Viscosity modifying admixtures (VMA) may also be used to help reduce segregation and the sensitivity of the mix due to variations in other constituents, especially to moisture content. Other admixtures including air entraining, accelerating and retarding may be used in the same way as in traditional vibrated concrete.

Choice of admixture for optimum performance may be influenced by the physical and chemical properties of the binder/ addition. Factors such as fineness, carbon content, alkalis and tricalcium silicate (C_3A) also have influence on the dosage of admixture. It is therefore recommended that compatibility is carefully checked if a change in supply of any of these constituents is to be made.

In this research work, commercially available super plasticizer Glenium B233, manufactured by BASF Construction Chemicals (India) Pvt. Ltd. is used for producing laterised self compacting concrete. This admixture is tailored to produce self compacting concrete. This is Polycarboxylic Ether (PCE) based and the recommended dosage is 0.545 to 1.635 % by weight of powder content. The physical and chemical properties of Glenium B-233 as per the manufacturer are listed in Table 3.7 and satisfy Indian Standard Specifications [91].

Table 3.7 Typical properties of Glenium B-233

Sl. No.	Particulars	Value/Description (as per manufacturer)
1	Form	Liquid
2	Color	Light brown
3	Specific gravity (25°C)	1.2
4	Viscosity (25°C)	50 to 150 cps
5	Chloride ion content	$<$ 0.2%
6	pH	6 to 9
7	Dosage	500 ml to 1500ml per 100kg of cementitious material

3.2.7 Supplementary cementitious material

For the present study, fly ash and ground granulated blast furnace slag (GGBFS) were the two types of supplementary cementitious materials used.

3.2.7.1 Fly ash

Fly ash from Ennore Thermal Power Plant, Chennai supplied by M/s Hi-Tech Fly ash (INDIA) Private limited has been used for the present study. The physical and chemical properties of fly ash as provided by the manufacturer are tabulated in Table 3.8.

It can be seen from Table 3.7 that the fly ash used conforms to low calcium fly ash as per the BIS specification [92] and which is considered as class F fly ash as per ASTM standards [21].

Table 3.8 Physical and chemical properties of fly ash

Sl. No.	Particulars	Value*	Standard value as per BIS [92]
a) Physical parameters			
1	Fineness residue retained on 325 mesh sieve (44μm)	28.50%	Nil
2	Moisture content	0.130%	Nil
3	Specific gravity	2.05	Nil
b) Chemical properties			
1	$SiO_2+Al_2O_3+Fe_2O_3$	94.42%	$>$ 70%
2	Sulphate trioxide as SO_3	0.66%	$<$ 2.75%
3	Sodium Oxide as Na_2O	0.38%	$<$ 1.5%
4	Loss on Ignition	0.45%	$<$ 12%

*Values as per the supplier's data

3.2.7.2 Ground granulated blast furnace slag

The slag from Nippon Denro Ispat Ltd., India, supplied by M/s CVC Associates, Cochin, has been used for the present study. The physical and chemical properties of ground granulated blast furnace slag as provided by the manufacturer are tabulated in Table 3.9.

It has been observed that from Table 3.9 that this ground granulated blast furnace slag can be used for the production of concrete as per the specification laid by Bureau of Indian Standards [93].

Table 3.9 Physical and chemical properties of ground granulated blast furnace slag (GGBFS).

Sl. No.	Particulars	Value*
a)Physical parameters		
1	Blaine fineness (m^2/kg)	463
2	Specific gravity	2.52
b)Chemical parameters		
1	CaO	38.70%
2	SiO_2	35.22%
3	Al_2O_3	19.14%
4	MgO	4.42%
5	Fe_2O_3	0.40%
6	Na_2O	0.36%
7	K_2O	0.40%
8	Loss on ignition	0.76%

*Values as per the supplier's data

3.3 Test Methods

Tests conducted on concrete are broadly classified in to two; viz. tests on fresh concrete and that on hardened concrete. All the tests were carried out as per the recommendation of relevant standards laid by Bureau of Indian Standards [94, 95].

3.3.1 Tests on Fresh Concrete

Tests on fresh concrete are primarily meant to assess the workability of concrete. Workability may be defined as the property of concrete which determines the amount of useful internal work necessary to produce full compaction [96]. The strength of concrete of a given mix proportion affects seriously by the degree of compaction; it is vital, therefore, that the consistency of the mix be such that the concrete can be transported, placed and finished sufficiently with ease and without segregation.

3.3.1.1 Slump test

It is always better to express workability in one of the parameters namely slump, compacting factor or flow, as these units are not always compatible with each other. Among different methods of measurement for workability, slump test is the most commonly used method which is actually meant for measuring consistency of concrete. In the present study, only slump test and compaction factor test have been conducted for the measurement of workability. Slump test is conducted for determining the consistency of concrete where the nominal maximum size of aggregates does not exceed 20mm. The slump test is very useful in detecting variations in the uniformity of a mix of given nominal proportions. Fig.3.5 shows the test setup for measurement of slump in concrete.

3.3.2 Tests on Fresh SCC

The key rheological parameters 'plastic viscosity' and 'yield value' mainly determine the filling ability of self compacting concrete. Slump flow and V-funnel tests demonstrate the best correlation with these, as well as having acceptable to good repeatability and reproducibility. Furthermore, the slump flow equipment is widely used in concrete practice, and the method is very simple and straight forward. Thus T_{500} combined with V-funnel test has been selected as the method for assessing the filling ability of SCC.

Figure 3.5 Test setup for slump measurement

The passing ability of fresh SCC can be tested by L-box or U-box or J-ring. There is some, but not very good, correlation between their results. The repeatability and reproducibility are acceptable to good for all the tests. For the L-box test a long practical experience was available, which led to a well-documented blocking criterion and correlation with the behavior in real construction elements has been shown to be good. For the J-ring, no clear information is available on the blocking criterion, but it could be a potential method for combining the measurement of the different properties of filling and passing ability. After a detailed evaluation, the EFNARC consortium selected both L-box and J-ring as the test methods for passing ability with equal priority [35]. For the present study, L-box test has been considered for the determination of passing ability of SCC.

3.3.2.1 Slump flow + T_{500} test

The slump flow test aims at investigating the filling ability of SCC [35]. It measures two parameters: flow spread and flow time (T_{500}). The former indicates the free, unrestricted deformability and later indicates the rate of deformation within a defined flow distance. The test procedure to measure slump flow+ T_{500} used was based on the European guidelines for self compacting concrete, specification, production and use [35] and brief description of the same is given in appendix A.

3.3.2.2 L-box test

The L-box test aims at investigating the passing ability of SCC [35]. It measures the reached height of fresh SCC after passing through the specified gaps of steel bars and flowing within a defined flow distance. With this reached height, the passing or blocking behavior of SCC can be estimated.

The test procedure to measure passing ability was based on the European guidelines for self compacting concrete, specification, production and use [35] and brief description of the same is given in appendix A.

3.3.2.3 V-funnel test

The V-funnel flow time is defined as the time period of SCC needed to pass through a narrow opening and gives an indication of the filling ability [35]. Passing this test ensures that SCC do not have blocking and/or segregation problems. The flow time of the V-funnel test is to some degree related to the plastic viscosity.

The test procedure to measure filling ability was based on the European guidelines for self compacting concrete, specification, production and use [35] and brief description of the same is given in appendix A.

3.3.3 Tests on Hardened Concrete

3.3.3.1 Common physical tests

The common physical tests on hardened concrete include compressive strength tests, split tensile test, flexural test and test for modulus of elasticity. Method of casting, curing and testing has been carried out as per the relevant Indian standard code of practices [94, 95].

Standard cubes of size 150mm and cylinders of size 150mm diameter and 300mm long specimens were considered for the determination of compressive strength of concrete. Standard cylinder of size 150mm diameter and 300mm long has been used for the determination of cylinder split tensile strength as well as for the determination of modulus of elasticity of concrete. In the case of flexural strength determination, standard beam specimen of size 500mm×100mm×100mm has been used.

3.3.3.2 Shear strength test

There is different experimental setup to determine shear strength of concrete under Mode II failure. It is practically difficult to get consistent result due to the difficulty in achieving ideal test conditions. Very limited literature is available in the area of tests on Mode II failure in concrete.

An experimental setup has been devised for the determination of shear strength of concrete in the present study. Figure 3.6 shows the photograph of the shear test device fabricated.

Typically, a specimen of size 250mm×100mm×100mm is clamped firmly over a length of 125mm with the help of nut and bolt arrangements in such a way that there is a gap of about 12mm at the bottom of the other half. This projecting portion of the specimen is then loaded from top with the help of

Figure 3.6 Shear testing apparatus developed for testing shear

a plunger of cross sectional area 125mm×100mm in such a way that a shear failure plain is expected at the mid length of the specimen. The load shall be applied continuously and without shock until the specimen fails. Figures 3.7 and 3.8 shows the experimental setup, showing the shear specimen before and after failure.

3.3.4 Heating of Specimen

To determine the performance of concrete exposed to high temperature, three test methods are commonly referred in most experimental programs. These are named as stressed, unstressed and unstressed residual strength tests [97]. In stressed tests, a pre load (20 to 40% of the compressive strength at 28°C) is applied to the specimen prior to heating and is sustained during the heating period. Heat is applied at a constant rate

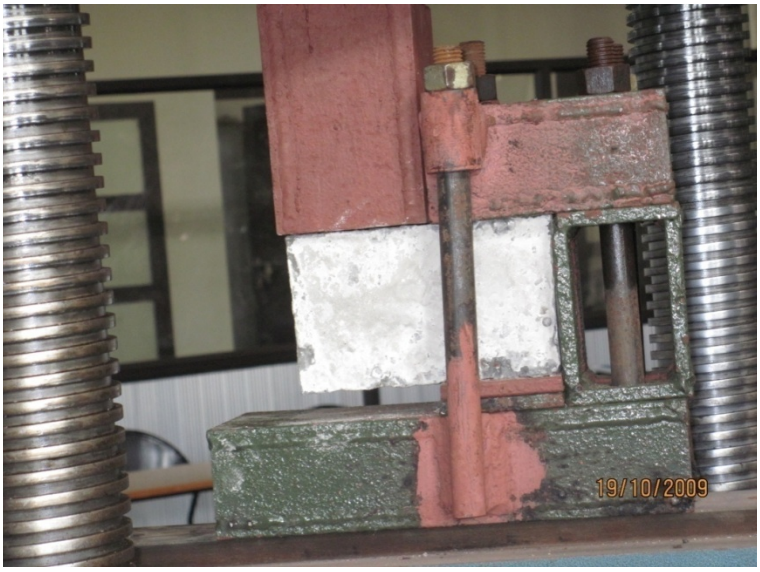

Figure 3.7 Shear specimen ready for loading

Figure 3.8 Shear specimen after failure

until a target temperature is reached, and is maintained for a time until a thermal steady state is achieved. Stress or strain is then increased at a prescribed rate until the specimen fails. In the unstressed test, the specimen is heated, without preload, at a constant rate to the target temperature, which is maintained until a thermal steady state is achieved. Stress or strain is then applied at a prescribed rate until failure occurs. In unstressed residual strength test, the specimen is heated without preload at a prescribed rate to the target temperature, which is maintained until a thermal steady state is reached within the specimen. The specimen is then allowed to cool, following a prescribed rate, to room temperature. Load or strain is applied at room temperature until the specimen fails.

In the present study, unstressed residual strength test method is used to obtain the effects of elevated temperature on various physical properties of concrete. An electrically heated furnace has been used to heat the test specimen to the required temperature levels. Inside of the furnace is cylindrical in shape and has a dimension of 400mm diameter and 600mm depth. The maximum temperature limit of the furnace is 1100ºC and 3 to 4 specimen could be placed in the oven at a time.

The temperature of the furnace can be controlled using a programmable temperature controller and the rate of increase in temperature was kept in line with the standard temperature rise–time curve recommended by Indian Standards which is the same as other International Standards [98, 99].

Equation 3.1 shows the standard temperature rise-time specified and the corresponding curve is shown in Figure 3.9.

$$T_1 - T_0 = 345 \log_{10}(8t_1+1) \quad \text{.......} \quad (3.1)$$

All specimens were taken out of the curing tank on 27th day. The surface of specimen were wiped with a dry cloth and kept inside the laboratory for 24 hours under room temperature. On the 28th day, they were heated to different temperature levels (200^{o}C, 400^{o}C and 600^{o}C) in the furnace

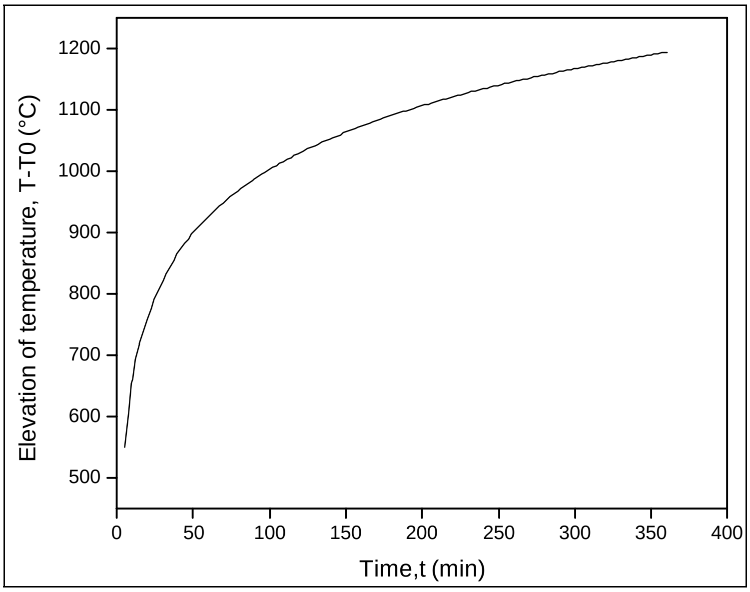

Figure 3.9 Standard temperature rise-time curve [98]

The photograph of furnace with specimens before heating and immediately after attaining required temperature are shown in Figures 3.10 and 3.11.

Figure 3.10 photograph of furnace with specimens kept ready for heating

Figure 3.11 Photograph of furnace immediately after reaching required temperature.

Chapter-4

STUDY ON MECHANICAL PROPERTIES OF LATERISED CONCRETE

4.1 Introduction

This chapter deals with the preliminary study on the suitability of laterite aggregate in concrete. Laterised concrete has been developed by replacing fine aggregate as well as by replacing both fine and coarse aggregate from conventional concrete. The properties of laterised concrete at fresh and hardened stages have been compared with the corresponding properties of conventional concrete. To study the influence of the source of laterite aggregate, concrete has been using laterite aggregate collected from various sources and their properties were compared. The influence of type of cement and supplementary cementitious material on laterised concrete has been also studied in this chapter.

4.2 Preliminary Study

Even though past researchers have used laterite as a replacement of fine aggregate for preparing concrete [1, 4, 5, 6, 8, 12, 14], study on concrete with coarse or all–in aggregate has not been reported yet. Literature survey reveals that the laterite is a material, when in-situ, it is soft enough to be cut with a knife. Having on exposure to the atmospheric action it gets hardened [1, 2]. Hence weathered laterite aggregate was considered for the present study. A preliminary study carried out on weathered laterite aggregate collected from

Cochin(CUSAT) proved that the aggregate satisfy the strength and grading characteristics of all-in aggregate specified for the use in concrete as per Indian standard [90]. Hence weathered laterite aggregate collected from the surface of laterite deposit has been considered for the present investigation.

Considering the fact that only normal strength concrete could be achieved when marginal materials like laterite is used in concrete, it has been decided to consider M25 grade concrete for the present investigation. The control concrete with crushed granite as coarse aggregate, river sand as fine aggregate and 53 Grade OPC has been referred as CC. The quantity of materials required for $1m^3$ of CC is given in Table 4.1. A typical mix design details of CC is presented in Appendix-B.

Table 4.1 Materials required for 1 m^3 of control concrete.

Sl. No.	Materials	Quantity of materials for $1m^3$ of Concrete
1	Cement(kg)	375
2	Fine Aggregate(kg)	639
3	Coarse Aggregate(kg)	1288
4	Water(L)	157.8
5	Super Plasticiser(L)	2.25

In order to select the type and source of laterite aggregate for the study, the following factors have been considered,

- Partial replacement of fine aggregate in concrete with weathered laterite fine aggregate.
- Total replacement of aggregate in concrete with weathered laterite all-in aggregate.

One group of laterised concrete has been prepared by replacing fine aggregate at different percentages (20, 40, 60, 80 and 100) with weathered laterite fine aggregate by volume and is designated as LCF. The corresponding concrete for various replacement levels of fine aggregate (sand) by laterite fine aggregate has been designated as LCF20, LCF40, LCF60, LCF80 and LCF100.

The other group of laterised concrete was prepared by replacing both aggregates from control concrete with weathered laterite all-in aggregate and has been designated as LCA. To study the influence of source of weathered laterite aggregate in concrete, aggregate was also collected from a wide range of locations in Kerala state.

Laterised concrete prepared using the aggregate collected from various sources have been designated as LCAQ, LCAC, LCAM and LCAK respectively, for the aggregates collected from Kollam, Cochin, Malappuram and Kasaragod.

4.2.1 Fresh Properties of Concrete

The properties of fresh concrete are studied in terms of slump and compaction factor. The tests were done as per Indian Standards [94] and results have been compared with the Indian Standard Specifications [100].The workability properties of different types of concrete measured has been compared in Table 4.2, wherein each value is the average of three observations.

Table 4.2 shows that the workability of laterised concrete increases with the replacement level of fine aggregate by laterite fine aggregate. There is about 15% increase in slump value compared to CC, when fine aggregate is completely replaced with laterite fine aggregate. The increased slump value in laterised concrete (LC) could be due to the presence of more spherical particles in weathered laterite aggregate compared to the conventional fine aggregate used.

Table 4.2 Workability properties of concrete.

Sl. No.	Concrete group	Concrete designation	Properties			
			Slump(mm)	Standard deviation	Compaction factor	Standard deviation
1	CC	CC	78	2.65	0.88	0.03
2	LCF	LCF20	80	3.61	0.89	0.03
3		LCF40	90	2.65	0.90	0.03
4		LCF60	90	2.65	0.90	0.03
5		LCF80	90	5.29	0.90	0.01
6		LCF100	92	2.65	0.90	0.05
7	LCA	LCAQ	69	3.61	0.86	0.03
8		LCAC	75	4.36	0.88	0.02
9		LCAM	72	3.46	0.87	0.02
10		LCAK	71	2.44	0.90	0.05

However, when all-in aggregate is used, laterised concrete showed a lower slump value compared with CC. This behaviour is expected due to the fact that all-in aggregate concrete has less fine content compared to conventional concrete. It must be noted that there is no much difference in the slump value of concrete among the source of laterite aggregate used in the present study. The compaction factor presented in Table 4.2 also shows the similar behaviour.

4.2.2 Properties of Hardened Concrete

Standard specimens namely cube (150mm size), cylinder (150mm diameter and 300mm long), and beam (100mm×100mm×500mm) have been cast for all types of concrete under consideration. The specimens were stripped from the moulds after 24 hours and cured by immersing in water under controlled laboratory environment for 28 days before testing for various strengths.

Properties of hardened concrete, namely unit mass, water absorption, cube compressive strength, cylinder compressive strength, flexure strength, split tensile strength and modulus of elasticity of concrete were determined and compared with the relevant Indian Standard Specifications [95,100].

The test results reported are the mean of triplicate samples and the individual test test result do not vary more than 15% from mean. The test results of control concrete and laterised concrete (LC) are summarised in Tables 4.3 and 4.4.

From Table 4.3, it can be observed that, as the percentage of laterite fine aggregate content increases, the water absorption of concrete increases while, the strength properties decreases. Udeoeyo et al. [12] have also observed a reduction in strength properties as the laterite fine aggregate content is increased in concrete. They reported that the laterite which consists of quartz and granular aggregates of kaolinite clay particles weakly cemented by sesquioxide (Fe_2O_3 and Al_2O_3) has less compressive strength than the sand it replaces in the concrete matrix and hence the reduction in strength. In the present study, it could be seen that, there is a higher rate of reduction in compressive strength for concrete with laterite aggregate content more than 60% in laterised concrete.

Table 4.3 Test results of LCF series.

Sl. No.	Concrete type (LCF Series)	Parameters	Properties						
			Unit mass (kg/m³)	Water absorption (%)	Cube compressive strength (MPa)	Cylinder compressive strength (MPa)	Flexural strength (MPa)	Split tensile strength (MPa)	Modulus of elasticity (MPa)
1	CC	Value	2470	1.10	34.51	29.50	5.47	4.36	34968
2		Standard deviation	9.17	0.00	0.22	1.03	0.56	0.05	1519.54
3	LCF20	Value	2468	1.10	33.80	28.92	4.81	4.28	34896
4		Standard deviation	6.24	0.04	1.12	1.03	0.69	0.24	2809.64
5	LCF40	Value	2465	1.00	33.09	28.29	4.38	2.29	34512
6		Standard deviation	6.24	0.02	2.14	2.20	0.35	0.29	978.54
7	LCF60	Value	2458	1.20	31.79	26.18	3.47	3.89	33619
8		Standard deviation	6.00	0.04	1.79	2.95	0.15	0.29	818.17
9	LCF80	Value	2458	1.20	28.21	25.32	3.21	3.77	32993
10		Standard deviation	2.65	0.07	1.87	2.13	0.18	0.30	655.27
11	LCF100	Value	2458	1.20	26.43	19.81	5.38	2.24	27502
12		Standard deviation	6.56	0.04	1.83	2.12	0.53	0.33	1104.78

Table 4.4 Properties of control concrete and LCA concrete series.

Sl. No.	Concrete type (LCA Series)	Parameters	Properties						
			Unit mass (kg/m^3)	Water absorption (%)	Cube compressive strength (MPa)	Cylinder compressive strength (MPa)	Flexural strength (MPa)	Split tensile strength (MPa)	Modulus of elasticity (MPa)
1	CC	Value	2470	1.10	34.51	29.50	5.47	4.36	34968
2		Standard deviation	9.17	0.00	0.22	1.03	0.56	0.05	1519.54
3	LCAQ	Value	2398	1.24	29.78	24.64	3.61	3.01	26228
4		Standard deviation	53.84	0.12	0.81	2.40	0.17	0.10	785.09
5	LCAC	Value	2440	1.24	31.41	25.99	4.20	2.93	28893
6		Standard deviation	24.88	0.17	0.76	1.84	0.29	0.42	2196.31
7	LCAM	Value	2385	1.24	30.58	25.99	4.18	3.00	27322
8		Standard deviation	29.14	0.08	1.76	0.94	0.27	0.19	1199.21
9	LCAK	Value	2380	1.25	27.59	24.60	3.53	2.82	25101
10		Standard deviation	15.59	0.18	0.79	1.71	0.26	0.07	1131.86

From Table 4.4, it could be seen that the source of weathered laterite all-in aggregate has less influence on properties of laterised concrete. As a result, the laterite aggregate available in Cochin (CUSAT), which is the nearest source has been considered for all further studies.

It has been observed that the weathered laterite fine aggregate obtained from various sources do not belong to any of the grading zone as per the Bureau of Indian Standard Specifications [90]. As a result, if fine aggregate is to be used in concrete, proper proportioning is to be carried out, which is a laborious process and involves wastage of money and material. On the other hand, the grading of weathered aggregate collected from all the sources satisfies the specifications for 20mm nominal size all-in aggregate grading as per Bureau of Indian Standard Specifications [90]. The particle size distribution curve of all-in laterite aggregate collected from the four sources has already been presented in Figure 3.2 of Chapter 3. Hence, it has been decided to consider all-in weathered laterite aggregate for further study.

4.3 Influence of Cement and Supplimentary Cementitious Materials on Laterised Concrete

In the present investigation an attempt has been made to study the influence of type of cement and supplementary cementitious materials on the fresh and hardened properties of concrete made with weathered laterite all-in aggregate.

4.3.1 Effect of Grade of OPC on Laterised Concrete

Ordinary portland cement of grade 33, 43, and 53 have been used to study the influence of grade of OPC on the physical properties of laterised concrete. The control concrete with 33, 43 and 53 grade OPC have been designated as CC33, CC43 and CC53 respectively. The corresponding laterised concrete with all-in aggregate have been designated as LCAC33, LCAC43 and LCAC53.

Table 4.5 shows the various mechanical properties of concrete tested.

From Table 4.5, it can be seen that, in general laterised concrete shows less slump compared to control concrete. The compressive strength, tensile strength and modulus of elasticity of laterised concrete are also less compared to the respective values of control concrete.

It could further be observed that concrete with 53 grade OPC gave higher strength properties for both control and laterised concrete compared to 33 and 43 grade OPC. As a result, 53 grade OPC has been considered for all further studies.

4.3.2 Influence of Supplementary Cementitious Materials in Laterised Concrete

The influence of supplementary cementitious materials on properties of laterised concrete has been studied by considering fly ash and Ground Granulated Blast Furnace Slag (GGBFS) as partial replacement to OPC in concrete. The OPC has been replaced with supplementary cementitious materials (fly ash and GGBFS) in different percentages (by weight), varying from 10% to 35% with an interval of 5%. The control concrete is designated as CCFL and CCGG respectively for fly ash and GGBFS. The corresponding laterised concrete are designated as LCFL and LCGG. The first two letters CC in the above designation represents control concrete made of conventional aggregate and 53 grade OPC cement, designed for M25 grade concrete. Similarly the LC in the above designation represents laterised concrete made of laterite all-in aggregate collected from Cochin (CUSAT) and with 53 grade OPC. Table 4.6 compares cube compressive strength

Table 4.5 **Properties of control concrete and laterised concrete made with different grades of OPC.**

Sl. No.	Concrete type	Parameters	Properties								
			Slump (mm)	Compaction factor	Unit mass (kg/m^3)	Water absorption (%)	Cube compressive strength (MPa)	Cylinder compressive strength (MPa)	Flexural strength (MPa)	Split tensile strength (MPa)	Modulus of elasticity (MPa)
1	CC33	Value	76	0.91	2410	1.13	28.83	25.04	3.58	4.97	25015
2		Standard deviation	7.21	0.03	25.51	0.13	1.07	0.68	0.40	0.46	3381.13
3	LCAC33	Value	68	0.89	2342	1.24	24.81	22.78	2.89	3.60	24201
4		Standard deviation	6.24	0.03	39.95	0.17	0.74	0.79	0.29	0.17	2726.84
5	CC43	Value	72	0.90	2399	1.16	31.29	26.93	3.64	5.10	27023
6		Standard deviation	4.58	0.05	28.85	0.11	2.51	3.79	0.32	0.25	1853.92
7	LCAC43	Value	68	0.89	2342	1.24	27.91	24.40	2.89	3.97	26735
8		Standard deviation	6.24	0.05	19.47	0.10	2.23	2.14	0.20	0.34	936.44
9	CC53	Value	78	0.88	2470	1.10	34.51	29.50	5.47	4.36	34968
10		Standard deviation	2.65	0.03	9.17	0.00	0.22	1.03	0.56	0.05	1519.54
11	LCAC53	Value	75	0.88	2440	1.24	31.40	25.99	4.20	2.93	28893
12		Standard deviation	4.36	0.02	24.88	0.17	0.76	1.84	0.29	0.42	2196.31

of concrete made with different percentage replacement of cement by fly ash and GGBFS.

Figures 4.1 and 4.2 depict the variation of cube compressive strength of CC and LC with various replacement level of cement by fly ash and GGBFS respectively.

From Table 4.6 and Figures 4.1 and 4.2, it could be observed that up to a certain percentage level of replacement (for fly ash 20% and GGBFS 25%), there is no much reduction in compressive strength of concrete and beyond which, there is a drastic reduction in strength of concrete. Rapid reduction in strength after a certain level of replacement of pozzolana has been reported elsewhere [57]. It could be observed from Figures 4.1 and 4.2 that, 20% fly ash and 25% GGBFS are the optimum replacement level for cement in both CC and LC for the economic production of concrete. Narayana and Swamy [57] also reported 20% to 30% is the economical replacement level for fly ash in concrete.

Chen and Liu [101] reported an initial increase in compressive strength in concrete when pozzolana is added at lower percentage of level of replacement. A similar behaviour was observed by Atis and Bilim [102]. Nasser and Marzouk cited by Poon et al.[103] reported that the increase in strength for the concrete containing pozzolana is due to the formation of tobermorite (a product of lime and pozzolana), which was reported to be two to three times stronger than the CSH gel.

However, the reason for this behavior cannot be generalised, as the pozzolanic activity of pozzolanic materials are greatly influenced by [24];

- The amount and composition of the glassy phase present.

Table 4.6 **Cube compressive strength of concrete with different replacement level of supplementary cementitious materials**

Sl. No.	Percentage replacement	Cube compressive strength (MPa)							
		CCFL		LCFL		CCGG		LCGG	
		Value	Standard deviation	Value	Standard deviation	Value	Standard deviation	Value	Standard deviation
1	0	34.51	0.81	31.41	1.23	34.51	2.08	31.40	2.39
2	10	34.40	2.61	31.11	1.21	34.31	3.04	31.32	1.34
3	15	34.19	1.16	30.99	1.43	34.19	2.03	31.14	0.99
4	20	34.19	1.69	30.99	2.41	33.89	0.78	31.11	2.17
5	25	32.80	1.82	28.59	1.82	33.95	1.82	31.08	2.33
6	30	32.41	1.69	28.00	1.96	31.29	3.88	28.80	2.74
7	35	31.11	1.46	27.20	1.49	30.19	1.16	27.00	2.40

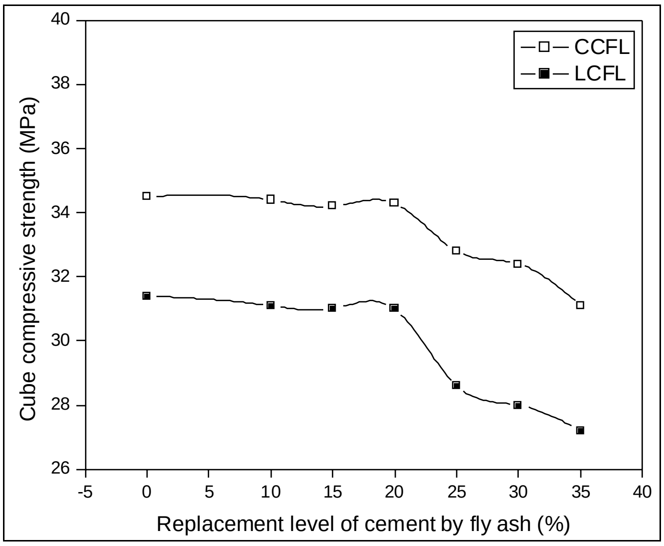

Figure 4.1 Variation of cube compressive Strength of CC and LC with various replacement level of cement by fly ash.

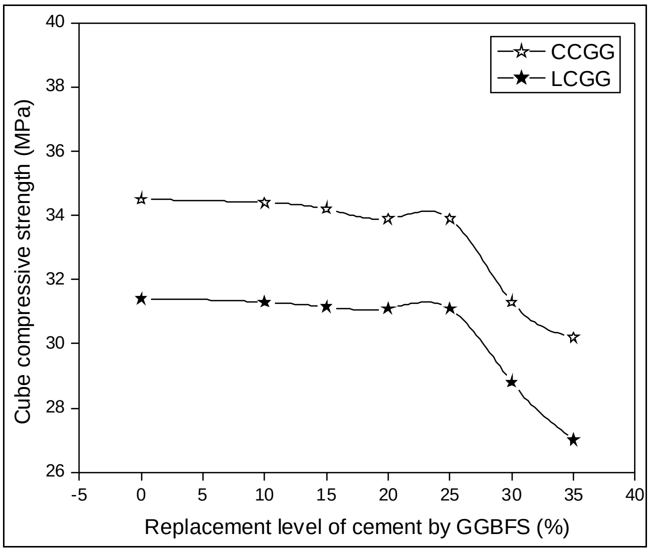

Figure 4.2 Variation of cube compressive Strength of CC and LC with various replacement level of cement by GGBFS.

- Mineralogical characteristics
- The quantity of particles under 45 microns size.
- Physical and chemical properties of cement used.

Since the present study yields 20% replacement of fly ash and 25% replacement of GGBFS for an economical concrete, further study on laterised concrete has been carried out with the said replacement level. The other properties of concrete with 20% replacement of cement by fly ash (CCFL20 and LCFL20) as well as with 25% replacement of cement by GGBFS (CCGG25 and LCGG25) are presented in Table 4.7.

From Table 4.7, it could be seen that the slump and compaction factor increased when supplementary cementitious materials are added in both control concrete and laterised concrete. The unit weight and cube compressive strength also increased and addition of GGBFS yielded higher increase in unit weight compared to fly ash. The changes in cube and cylinder compressive strength are only nominal with the optimum replacement level of pozzolans (20% fly ash and 25% GGBFS). A decrease in flexural strength is noticed except for laterised concrete with GGBFS. A similar trend was observed for split tensile strength and modulus of elasticity also.

4.4 Alkali-Silica Reaction

Alkali-silica reaction is a chemical process in which alkalis, mainly from the cement, combine with certain types of minerals in the aggregate, when moisture is present. This reaction produces a gel that can absorb water and expand to cause cracking and disruption of concrete. The amount of expansion and resulting damage that occurs in concrete due to alkali-silica reaction depends on the availability of alkali and silica in the system. The main source of alkali in concrete is portland cement. The silica content in laterite aggregate is comparatively less than crushed granite aggregate. The addition of mineral admixtures also has been shown to reduce the expansion of concrete affected by alkali-silica reaction. Hence, laterised concrete is expected to perform better than conventional concrete and laterised concrete with mineral admixture should perform still better.

Table 4.7 Physical and mechanical properties CCFL, LCFL, CCGG and LCGG

Sl. No.	Concrete type (LCF Series)	Parameters	Properties								
			Slump (mm)	Compaction factor	Unit mass (kg/m^3)	Water absorption (%)	Cube compressive strength (MPa)	Cylinder compressive strength (MPa)	Flexural strength (MPa)	Split tensile strength (MPa)	Modulus of elasticity (MPa)
1	CC	Value	78	0.88	2470	1.10	34.51	29.50	5.47	4.36	34968
2		Standard deviation	2.65	0.03	9.17	0.00	0.22	1.03	0.56	0.05	1519.54
3	LC	Value	75	0.88	2440	1.24	31.41	25.99	4.20	2.93	28893
4		Standard deviation	4.36	0.02	24.88	0.17	0.76	1.84	0.29	0.42	2196.31
5	CCFL20	Value	100	0.92	2486	0.60	34.19	30.94	4.88	3.77	30505
6		Standard deviation	7.81	0.03	24.33	0.07	1.69	3.33	0.55	0.33	1028.83
7	LCFL20	Value	89	0.92	2440	0.66	30.99	29.73	4.29	2.90	28200
8		Standard deviation	7.81	0,03	7.21	0.07	2.41	2.56	0.17	0.17	1447.51
9	CCGG25	Value	95	0.92	2480	0.72	33.95	28.94	5.16	3.99	31660
10		Standard deviation	4.36	0.02	6.93	0.09	1.82	1.33	0.59	0.50	1323.18
11	LCGG25	Value	84	0.92	2460	0.78	31.08	27.92	4.48	3.24	29358
12		Standard deviation	6.08	0.05	11.14	0.05	2.33	1.72	0.11	0.16	1353.13

4.5 Concluding Remarks

Based on the present study, it may be noted that, weathered laterite fine aggregate obtained from various sources do not belong to any of the grading zone, and as result, if fine aggregate is to be used in concrete, proper proportioning is to be carried out, which is a laborious process and involves wastage of money and material. On the other hand, the grading of weathered aggregate collected from all the sources satisfies the specifications for 20mm nominal size all-in aggregate grading. Hence, use of all-in weathered laterite aggregate in concrete is advisable for making laterised concrete. Weathered laterite aggregate has been collected from a wide range of places and its influence on strength and other parameters have been compared. It has been observed that there is no much difference in the mechanical properties of laterised concrete made with weathered laterite all-in aggregate from different sources.

Concrete with 53 grade OPC gave higher strength properties for both control and laterised concrete compared to 33 and 43 grade OPC. Twenty percent replacement by fly ash and 25% replacement by GGBFS have been found as economical replacement levels for both conventional and laterised concrete.

Chapter-5

LATERISED SELF COMPACTING CONCRETE

5.1 Introduction

In places where compaction and placing are difficult, (such as jacketing of structural elements, back filling near retaining structures etc.) Self Compacting Concrete (SCC) is a better choice than conventional concrete. Even though limited study has been carried out in the area of laterised concrete [10, 11, 12, 104, 105,106], no study has been reported in the case of Laterised Self Compacting Concrete (LSCC). There is a need to have a mix design methodology for making LSCC. This chapter proposes a mix design methodology for LSCC for normal strength concrete (M20 to M40 grade) and it's validation by conducting experiments.

5.2 Mix Design Methodology

Self compatibility can be largely affected by the characteristics of materials and the mix proportion. A rational mix design method for self compacting concrete was initially proposed by Okumura [107]. Even though several researchers have proposed mix design procedures for SCC [23, 43, 108, 109], Nan Su et al. [110] proposed a simplified method based on Okumura's method. Karjini [111] observed that the cement content arrived using Nan Su method is not sufficient for a uniform mix and proposed modified Nan Su method.

In the present investigation, modified Nan Su method has been considered as a base for the design of laterised SCC. It may be noted that the mix design procedures available in literature are suitable for making SCC with conventional aggregate. However, SCC using laterised aggregate requires more powder content for a uniform flow due to the fact that, weathered laterite all-in aggregate contains less fine materials compared to aggregate used for SCC.

Hence trial mixes have been made for M20, M25, M30, M35 and M40 grade concrete. To arrive at the various proportions of the constituent materials in each grade of concrete, several trial mixes have been made and based on the observations such as filling ability, stability, strength properties etc., the proportion of each grade of concrete have been finalized. The final proportion of different ingredients and their relationship with characteristic strength are presented in Appendix C.

Based on the results of the above said study, a mix design methodology has been proposed for LSCC of grade M20 to M40 and its details are presented in the sessions 5.2.1 to 5.2.5

5.2.1 Determination of Cement Content

Modified Nan Su method [111] proposes equation 5.1for the determination of cement content in SCC.

$$C_w = M_f\,[f_{ck} / 0.14] \quad \text{.......} \quad (5.1)$$

However, it was observed that the above modification factor M_f proposed for the calculation of cement content, yields less cement requirement for a cohesive LSCC. It has been reported elsewhere also that SCC with marginal materials as aggregates requires more powder content to achieve uniform flow [112,113]. Hence based on trial mixes, a different modification factor has been proposed for LSCC and the same is presented as equation 5.2.

$$M_f = (f_{ck}^2 / 1400) - (f_{ck} / 12) + 3.75 \quad \text{.......} \quad (5.2)$$

Table 5.1 compares the proposed modification factor for LSCC with the modification factor of modified Nan Su method for SCC. The comparison is given for concrete grades from M20 to M40.

5.2.2 Determination of the Quantity of Additions

Commonly used additions in SCC to improve various properties like flowability, cohesion, segregation resistance, bleeding, settlement etc., is fly ash and ground granulated blast furnace slag [48, 114]. Because of less fine materials in laterite all-in aggregate, large quantity of additional fine materials is required for making LSCC, compared to SCC. Accordingly, the quantity of additions to be used in LSCC can be determined using the relation 5.3.

Table 5.1 Comparison of modification factor

Sl. No.	Grade of concrete	Modification factor	
		Proposed for LSCC	Modified Nan Su method for SCC
1	M20	2.37	2.360
2	M25	2.11	2.020
3	M30	1.89	1.750
4	M35	1.71	1.535
5	M40	1.56	1.380

$F_w = C_w [S_a / S_c] F_a$ ------------------- (5.3)

The additions factor F_a for LSCC has been proposed based on trial mixes and can be determined using equation 5.4.

$F_a = 1.74 - [f_{ck} / 100]$ ....(5.4)

The F_a has been arrived at based on the fact that, quantity of additions required for LSCC is less for higher grades of concrete, as higher grade of concrete has more cement content than lower grade.

5.2.3 Calculation Water Powder Ratio

The mixing water content required is an important parameter in mix design and is the total amount of water required for cement and filler. For f_{ck} ranging from 20 to 40 MPa, the water to powder ratio (*W/P*) may be calculated using the equation 5.5., which has been proposed based on experimental study.

$$(W/P) = 0.4 - [2.5 / 1000]\, f_{ck} \quad \text{...(5.5)}$$

5.2.4 Calculation of Aggregate Content

Laterite all-in aggregate of 12.5 mm nominal size has been considered for the present study. It has been reported elsewhere that the air content in SCC ranges between 1% and 1.5% [110]. In LSCC, the air content has been taken as 1% of the total volume of concrete for the calculation of aggregate content. Having obtained all other ingredients for LSCC, the aggregate content can be calculated based on the volume ratio equilibrium equation. Accordingly, the equation 5.6 can be used for the calculation of aggregate for one cubic meter of LSCC.

$$1000\,(1-V_a) = [Cw / S_c] + [Fw / S_a] + [W_l / S_l] + W_w \quad \text{....................(5.6)}$$

5.2.5 Superplasticiser (SP) Dosage

Addition of optimum amount of SP will improve the flowability, self compacting ability, water demand and segregation resistance of LSCC. The dosage of SP primarily depends on the manufacturer's guidelines and shall be adjusted with trial mix. It is important to note that the water content in SP should be adjusted in the total quantity of mixing water.

5.3 Validation of Mix Design Procedure

In order validate the mix design procedure for LSCC, mixes ranging from M20 to M40 have been designed using the proposed methodology and tests have been carried out. Table 5.2 presents the list of mixes designed and various ingredients required for 1m^3 of LSCC with fly ash as addition.

Table 5.2 Mix proportion for 1 m^3 of LSCC with fly ash as addition

Sl. No.	Mix	Cement (kg)	Fly ash (kg)	Laterite aggregate (kg)	Total water (L)	SP (L)
1	M20	338.6	338.9	1299.5	237.0	6.8
2	M25	373.2	361.5	1204.9	249.8	7.4
3	M30	400.7	375.1	1188.4	240.5	7.8
4	M35	420.0	379.5	1140.0	249.9	8.0
5	M40	431.4	375.8	1156.2	242.2	8.1

Sub-sections 5.3.1 and 5.3.2 presents the test results of LSCC at fresh stage and hardened stage respectively.

5.3.1 Properties of Fresh LSCC

The European guidelines for self compacting concrete [33] were the first to codify the various testing methods on SCC. As there is no specific test method suggested by BIS for SCC, all tests on LSCC at fresh stage have been carried out based on the guidelines given in 'The European guidelines for Self Compacting Concrete' [33].

The slump flow and T_{500} time are tests to assess respectively the flowability and flow rate of SCC in the absence of obstructions, Figure 5.1 depicts typical flow pattern of M20 grade LSCC.

V-funnel test used to assess the viscosity and filling ability of LSCC has been considered for deciding the viscosity class of SCC in the present study. Figure 5.2 shows typical experimental setup for V-funnel test of LSCC.

The L-box test is used to determine the passing ability of SCC against obstructions without

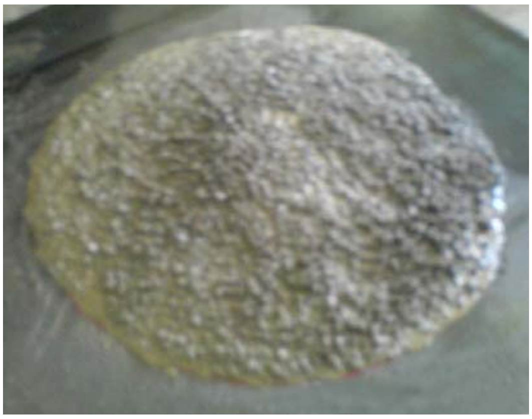

Figure 5.1 Flow pattern of M20 grade LSCC (Typical)

Figure 5.2 V- Funnel Test for M30 grade LSCC (Typical)

causing segregation or bleeding. Figure 5.3 shows the L-box test setup with 3 rebars for LSCC.

Table 5.3 presents the test results of LSCC with fly ash as additions at fresh stage and Standard Deviation (SD). Each value is the average of three test results.

Figure 5.3 L-Box test for M25 grade LSCC (Typical)

From Table 5.3, it can be seen that the slump flow value obtained is within the range from 660mm to 750mm and hence all the grades of LSCC made belongs to the slump flow class SF2 [35]. This means that the proposed mix design method for LSCC can be used for making concrete for many normal applications. (Refer section 3.3.2).

The V-funnel value is considered for deciding the viscosity class of SCC. From Table 5.3, it can be observed that the viscosity class of all grades of LSCC made falls under VS2/VF2 [35] (Refer section 3.3.2 of Chapter 3).

Table 5.3 further shows that, with 3 rebars, the passing ratio of all grades of LSCC is more than 0.8. So, the passing ability of LSCC comes under PA2 class [35]. Hence this concrete can be used for construction of civil engineering structures (Refer section 3.3.2 of Chapter 3).

In order to validate the suitability of GGBFS as addition in LSCC, a typical mix of M25 grade has been designed using the proposed mix design methodology. The properties of LSCC with GGBFS at fresh stage observed have been presented in Table 5.4 and the same has been compared with the corresponding mix with fly ash as addition. From Table 5.4, it could be seen that the properties of LSCC at fresh stage for both fly ash and GGBFS as additions have closely comparable results.

From Tables 5.3 and 5.4, it could be concluded that the proposed mix design methodology for LSCC is acceptable for general use based on results of fresh stage.

Table 5.3 Properties of LSCC with fly ash as addition at fresh stage

Sl. No.	Grade of LSCC	Slump flow (mm)		T_{500} (s)		Tv (s)		Δh (mm)		Passing ratio
		Value	Standard deviation	Value	Standard deviations	Value	Standard deviation	Value	Standard deviation	
1	M20	700	4.51	6	0.58	17	1.00	75	3.46	0.824
2	M25	700	7.51	7	0.00	17	1.00	75	1.00	0.824
3	M30	680	8.19	6	0.00	23	1.73	70	2.65	0.869
4	M35	680	4.36	7	1.00	22	1.00	65	4.36	0.937
5	M40	685	6.08	5	0.00	24	1.00	65	1.15	0.937

Table 5.4 Comparison of properties of LSCC with fly ash and GGBFS as additions at fresh stage

Sl. No.	Property	M25 LSCC with			
		Fly ash as additions		GGBFS as additions	
		Value	Standard deviation	Value	Standard deviation
1	Slump flow (mm)	700	4.51	700	4.58
2	T_{500} (s)	7	0.00	7	0.00
3	Tv (s)	17	1.00	17	0.00
4	Δh (mm)	75	1.00	75	7.42
5	Passing ratio	0.824	--	0.824	--

5.3.2 Properties of Hardened LSCC

All tests on hardened LSCC have been carried out based on the relevant Indian Standard Specifications [13]. The results reported correspond to the average value of 3 specimens tested.

The observed strength has been compared with the target average compressive strength calculated [115]. The target average compressive strength at 28 days ($\bar{f}_{ck}$) has been arrived based on the equation 5.7.

$$\bar{f}_{ck} = f_{ck} + k \times s \quad] \text{ ---------- } (5.7)$$

Table 5.5 shows the cube and cylinder compressive strength of LSCC obtained for various grades of concrete.

Table 5.5 Compressive strength of LSCC with fly ash as addition

Sl. No.	Grade of LSCC	Cube compressive strength (MPa)			Cylinder compressive strength (MPa)		
		Experimental (f_{ck})	Standard deviation	Target mean strength($\bar{f}_{ck}$)	Experimental (f_{cy})	Standard deviation	f_{cy}/ f_{ck}
1	M20	28.77	1.56	26.60	22.41	1.71	0.78
2	M25	34.31	1.30	31.60	27.20	2.53	0.79
3	M30	41.91	2.54	38.25	32.80	3.44	0.78
4	M35	46.12	1.61	43.25	36.78	4.33	0.80
5	M40	51.03	3.52	48.25	40.78	4.13	0.80

It can be observed from the Table 5.5 that, the expected target cube strength of LSCC could be arrived for all grades of concrete (M20 to M40) prepared based on the proposed mix design methodology. The ratio of cylinder to cube compressive strength has also presented in Table 5.5. It may be noted that the average value of the above ratio for grades of LSCC from M20 to M40 is 0.79 and this is comparable with the commonly accepted value for normal strength concrete (0.80).

Table 5.6 presents the modulus of rupture and cylinder split value for LSCC.

Table 5.6 Modulus of rupture and split tensile strength for various grades of LSCC with fly ash as addition

Sl. No.	Grade of LSCC	Modulus of rupture (MPa)		Split tensile strength (MPa)	
		Value	Standard deviation	Value	Standard deviation
1	M20	4.20	0.24	2.94	0.19
2	M25	5.30	0.30	3.28	0.27
3	M30	6.40	0.11	3.45	0.43
4	M35	7.10	0.25	3.71	0.33
5	M40	7.80	0.36	4.01	0.29

It may be noted that the modulus of rupture value is higher than the cylinder split strength, which is an expected test result for any concrete. In the present study, while modulus of rupture of M20 grade LSCC showed 40% higher value, M40 grade LSCC showed 95% higher value compared to the respective split tensile strength. This variation is comparable with the report of ACI committee [42], where it has been stated that the modulus of rupture of concrete can be higher by 40% to 80% compared to split tensile strength.

Table 5.7 presents the modulus of elasticity of the LSCC tested. The theoretical value of modulus of elasticity based on the relation $5000\sqrt{f_{ck}}$ [100] has also been presented in Table 5.7.

Table 5.7 Modulus of elasticity of LSCC with fly ash as addition

Sl. No.	Grade of LSCC	Modulus of elasticity (MPa)		
		Experimental	Standard deviation	Theoretical
1	M20	28250	1703.10	26823
2	M25	30180	901.08	29292
3	M30	32019	1373.54	32365
4	M35	34294	1335.36	33956
5	M40	36081	3130.50	35714

It may be noted that the variation between the experimental and theoretical values of modulus of elasticity is between + 5% and -1% for the present study. So, it could be concluded that the relation $5000\sqrt{f_{ck}}$ can be used to predict the modulus of elasticity of LSCC for concrete grades between M20 and M40.

The strength properties (at 28th day) of M25 grade LSCC with GGBFS as addition (LSCCGG) has been compared with the corresponding LSCC with fly ash as addition (LSCCFL) in Table 5.8.

From Table 5.8, it could be seen that the results of M25 LSCC with GGBFS are comparable with corresponding results of LSCC with fly ash as addition. It may further be noted that compressive strength and modulus of elasticity of LSCC with GGBFS is even higher than that of LSCC with fly ash as addition. However, in the case of tensile strength, LSCC with fly ash has higher strength compared that having GGBFS as addition.

Table 5.8 Comparison of strength properties of LSCC with fly ash and GGBFS as additions

Sl. No.	Property	M25 LSCC with			
		Fly ash as addition		GGBFS as addition	
		Value	Standard deviation	Value	Standard deviation
1	Cube compressive strength (MPa)	34.31	1.30	37.23	1.61
2	Cylinder Compressive strength(MPa)	27.20	2.53	29.39	1.79
3	Flexural strength (MPa)	5.30	0.30	5.23	0.44
4	Cylinder split tensile strength (MPa)	3.28	0.27	3.65	0.31
5	Modulus of elasticity (MPa)	30180	901.08	31123	2252.07

5.4 Concluding Remarks

A new mix design methodology has been proposed for LSCC, which can be used for the design of concrete with weathered all-in laterite aggregate for grades ranging from M20 to M40.

- The method proposed can be used for fly ash as well as GGBFS as additions.
- Since weathered laterite all-in aggregate does not have sufficient fines, large quantity of additions is to be added in LSCC to get the required flow.
- The proposed mix design methodology has been validated by conducting experimental studies and observing results at fresh and hardened stages.
- The theoretical relation $5000\sqrt{f_{ck}}$ can be used to predict the modulus of elasticity of LSCC fly ash as addition.

Chapter -6

BEHAVIOUR OF LATERISED CONCRETE AT ELEVATED TEMPERATURE

6.1 Introduction

Influence of aggregate type, cement type and pozzolans on the physical and mechanical properties of laterised concrete at ambient temperature has been discussed in chapter 4.

This chapter discusses the influence of the above said parameters on the mechanical properties of laterised concrete when exposed to high temperature.

Laterised concrete specimens prepared by varying different parameters were heated to different temperature levels (200ºC, 400ºCand 600ºC). The specimens were then cooled to ambient temperature in two different ways, namely;

- by air cooling and
- by water cooling

All the specimens were tested at ambient temperature after cooling and the various physical properties have been compared with control concrete. The types of concrete considered for the present study include

- CC-Control concrete designed for M25 grade.
- LCF-Laterised concrete with laterite aggregate as fine aggregate designed for M25 grade.
- LCAC53-Laterised concrete with all-in laterite aggregate and OPC 53 grade cement designed for M25 grade.
- CCFL20-Control concrete in which 20% cement is replaced with fly ash.

- LCFL20-Laterised concrete designed for M25 grade with all-in laterite aggregate and 20% cement is replaced with fly ash.
- CCGG25-Control concrete in which 25% cement is replaced with GGBFS.
- LCGG25-Laterised concrete designed for M25 grade with laterite all-in aggregate and 25% cement is replaced with GGBFS.
- LSCCF-Laterised self compacting concrete with fly ash as addition.
- LSCCG- Laterised self compacting concrete with GGBFS as addition.

6.2 Compressive Strength

Tables 6.1 and 6.2 present the compressive strength of cubes and cylinders of all types of concrete at different test conditions and Standard Deviation (SD). Each value in the tables is the average of three test results.

It could be seen from Table 6.1 that, the cube compressive strength of both CC and LC decreases with increase in temperature, which is in agreement with the findings of earlier investigators [15, 71, 73, 74 and 116]. It may be noted that the strength reduction in concrete when exposed to high temperature is primarily due to the decomposition of the cement paste and the corresponding loss of adhesion [70]. It could be seen that water cooled concrete shows low residual strength compared to air cooled concrete. Similar behavior has been reported elsewhere also [15,116]. This is due to the fact that hydrated portland cement contains a significant amount of fire free calcium hydroxide and will decompose into calcium oxide due to loss of water at high temperature. If this calcium oxide is wetted after being cooled or kept in a moist environment, it transforms into calcium hydroxide again and causes a volume change. The concrete may crumble as a result of such changes in volume [117].

It could be seen from Table 6.2, that the cylinder compressive strength also decreases with increase in temperature and water cooled concrete has less strength than air cooled concrete.

Table 6.1 Cube compressive strength of concrete after the exposure to elevated temperature

Sl. No.	Concrete type (M25 grade)	Cube compressive strength (MPa)													
		Ambient (28°C)		Air cooled						Water cooled					
				Exposure temperature (°C)						Exposure temperature (°C)					
				200		400		600		200		400		600	
		Mean	SD	Mean	SD	Mean	SD	Mean	SD	Mean	SD	Mean	SD	Mean	SD
1	CC	34.51	0.22	32.00	0.85	28.51	1.98	17.91	2.05	30.49	2.36	27.50	1.48	16.99	1.87
2	LCF	26.43	1.83	21.37	0.88	18.68	1.59	13.56	1.00	18.15	0.86	15.30	1.15	12.58	1.06
3	LCAC53	31.41	0.76	28.51	0.81	25.67	1.51	19.50	1.49	27.50	2.00	22.47	1.05	18.50	1.42
4	CCFL20	34.19	1.69	31.23	2.12	29.81	3.01	28.80	1.76	30.63	1.95	28.80	1.80	27.08	1.82
5	LCFL20	30.99	2.41	28.63	1.69	27.23	2.59	26.14	2.88	27.59	1.48	26.32	1.95	24.98	3.34
6	CCGG25	33.95	1.82	31.46	2.20	29.96	2.57	28.62	1.08	31.20	1.97	29.45	1.23	27.44	0.98
7	LCGG25	31.08	2.33	29.36	1.79	28.42	2.90	26.98	3.26	28.59	1.73	26.91	2.30	25.07	1.53
8	LSCCF	34.31	1.30	33.21	1.57	32.10	2.88	30.90	1.70	32.38	3.10	31.70	1.75	30.10	1.46
9	LSCCG	37.24	1.61	34.28	3.15	30.98	4.34	27.62	1.69	33.51	2.45	29.87	1.91	25.55	0.84

Table 6.2 Cylinder compressive strength of concrete after exposure to elevated temperature

Sl. No	Concrete type (M25 grade)	Cylinder compressive strength (MPa)													
		Ambient 28°C		Air cooled						Water cooled					
				Exposure temperature(°C)						Exposure temperature(°C)					
				200°C		400°C		600°C		200°C		400°C		600°C	
		Mean	SD	Mean	SD	Mean	SD	Mean	SD	Mean	SD	Mean	SD	Mean	SD
1	CC	29.50	1.03	27.01	1.45	23.99	1.87	15.66	0.63	25.99	3.28	23.01	2.22	14.49	1.13
2	LCF	19.81	2.12	16.90	1.93	14.90	2.23	12.90	0.99	15.20	1.56	12.90	1.03	11.20	1.04
3	LCAC53	25.99	1.84	23.50	1.31	20.49	2.12	18.00	1.39	22.48	3.19	20.00	2.05	16.48	1.76
4	CCFL20	30.94	3.33	29.05	3.34	27.58	3.13	24.33	3.34	27.17	1.04	26.26	1.18	24.41	1.62
5	LCFL20	29.73	2.56	27.09	2.42	25.20	1.97	22.94	0.54	25.77	1.89	23.69	1.10	21.35	1.42
6	CCGG25	28.94	1.33	27.73	2.56	25.58	3.35	22.34	2.81	25.17	2.15	24.26	1.35	19.92	2.18
7	LCGG25	27.92	1.72	25.35	1.96	23.13	2.98	21.81	1.44	22.82	2.46	21.73	2.99	19.43	2.18
8	LSCCF	27.20	2.53	26.29	1.81	25.09	2.21	24.11	1.03	25.62	1.51	24.79	2.66	23.32	2.40
9	LSCCG	29.39	1.79	26.98	1.31	26.00	2.25	24.26	2.69	26.94	1.36	25.20	1.53	22.30	2.00

From Tables 6.1 and 6.2, it could be observed that the strength of concrete with supplementary cementitious material (CCFL20, LCFL20, CCGG25 and LCGG25) is higher compared with that of concrete without supplementary cementitious materials (CC, LCF and LCAC53). This is due to the formation of tobermorite gel (C-S-H phase), as a result of the reaction of mineral admixtures (FA and GGBFS) with $Ca(OH)_2$ [118]. At elevated temperature, concrete with supplementary cementitious materials (CCFL20, LCFL20, CCGG25 and LCGG25) performed better than concrete without supplementary cementitious materials (CC, LCF and LCAC53). This is in agreement with the report of Poon et al. [59]. The strength reduction of CC, LCF and LCAC53 with temperature is compared with their respective strength at ambient temperature in Figures 6.1 to 6.4. Figures 6.1 and 6.2 depicts the percentage reduction in cube compressive strength with temperature under air cooled and water cooled environment respectively. The corresponding comparison in the case of cylinder specimen is shown in Figures 6.3 and 6.4.

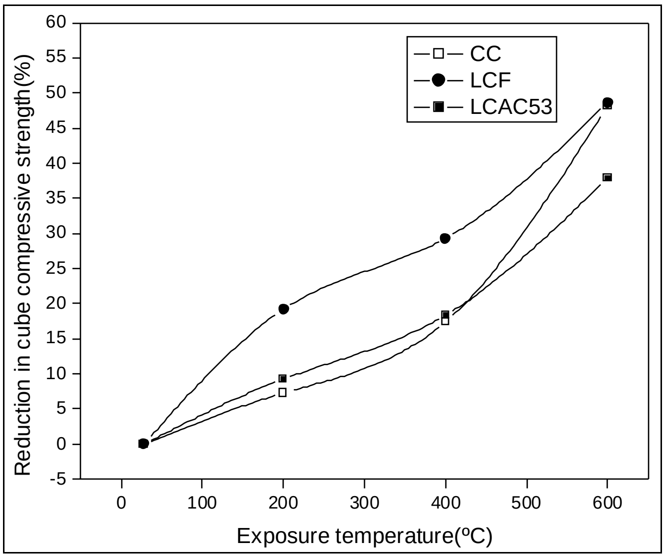

Figure 6.1 Percentage reduction in cube compressive strength of concrete with temperature after air cooling.

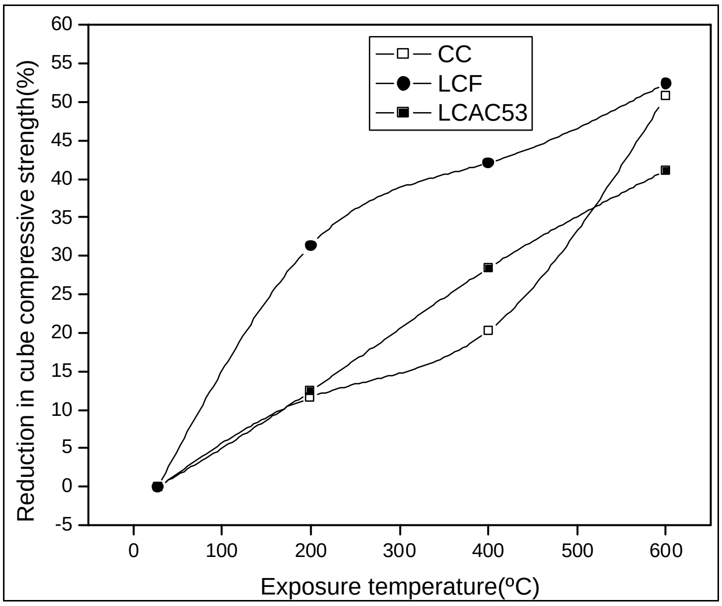

Figure 6.2 Percentage reduction in cube compressive strength of concrete w.th temperature after water cooling.

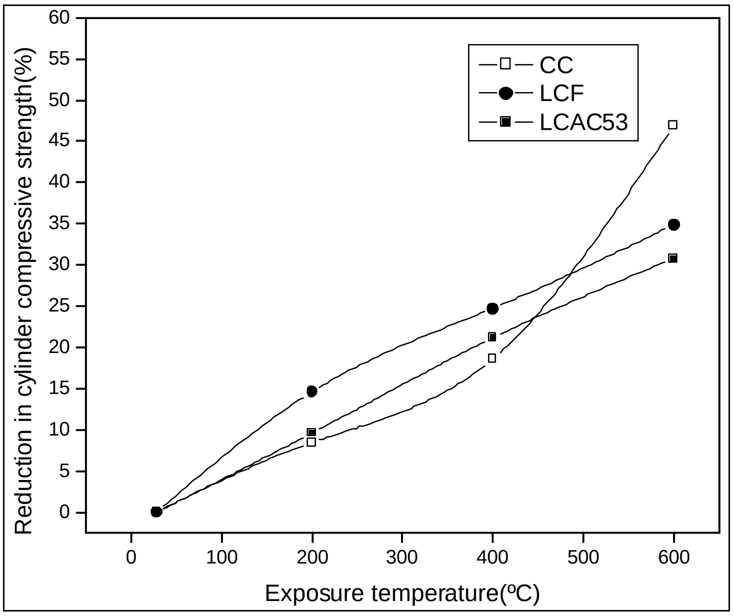

Figure 6.3 Percentage reduction in cylinder compressive strength of concrete with temperature after air cooling.

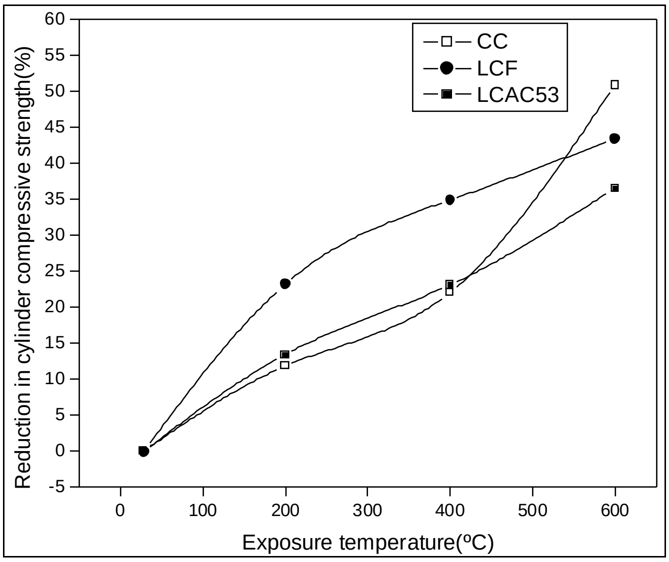

Figure 6.4 Percentage reduction in cylinder compressive strength of concrete with temperature after water cooling.

From Figures 6.1 to 6.4, it can be seen that the rate of reduction in compressive strength with temperature of CC, LCF and LCAC53 under two different cooling environments are more or less the same for both cube and cylinder specimen. However LCF shows slightly higher strength reduction compared to others. The rate of reduction in strength for CC and LCAC53 is almost linear but LCF shows a higher rate of strength reduction up to 200ºC and beyond this, the rate of reduction is similar to other concrete. This behavior could be explained as follows. The greater the amount of moisture lost, the lower is the strength. At lower temperatures, the amount of moisture lost could vary according to exposure duration. But at very high temperatures, most of the moisture would have been driven off irrespective of other variables. The volume changes can result large internal stresses and lead to micro cracking and fracture. Elevated temperatures also cause chemical micro structural changes, such as water migration, increased water dehydration, interfacial thermal incompatibility and chemical decomposition of hardened cement paste and aggregates in general. All these changes lead to a decrease in the strength and stiffness of concrete and an increase in deformations. [4]. In the present investigation, the reduction in cube strength for CC, LCF and LCAC53 at 600ºC is 29%, 37% and 32% under air cooling and 32%, 52% and 35% under water cooling environment respectively. The corresponding values for cylinder compressive strength is 29%, 31%and 31% for air cooled environment and 34%, 43% and 37% for water cooled environment.

Figurers 6.5 to 6.8 depict the percentage reduction in compressive strength of LCAC53, CCFL20 and LCFL20 at elevated temperature in both environments compared to the ambient compressive strength of respective concrete.

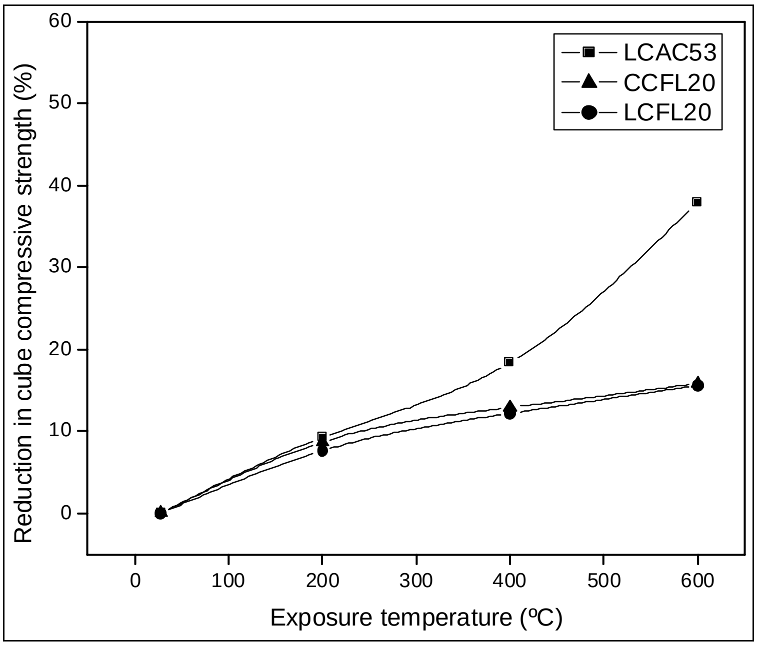

Figure 6.5 Percentage reduction in cube compressive strength of concrete with temperature after air cooling.

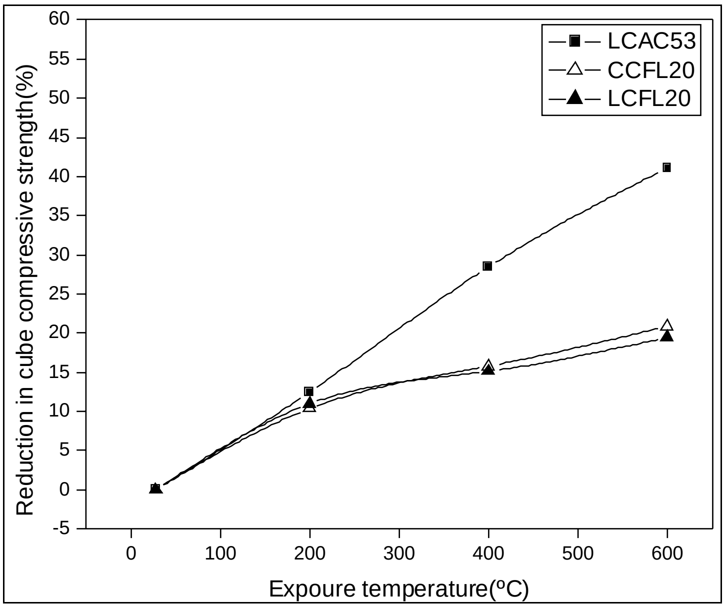

Figure 6.6 Percentage reduction in cube compressive strength of concrete with temperature after water cooling.

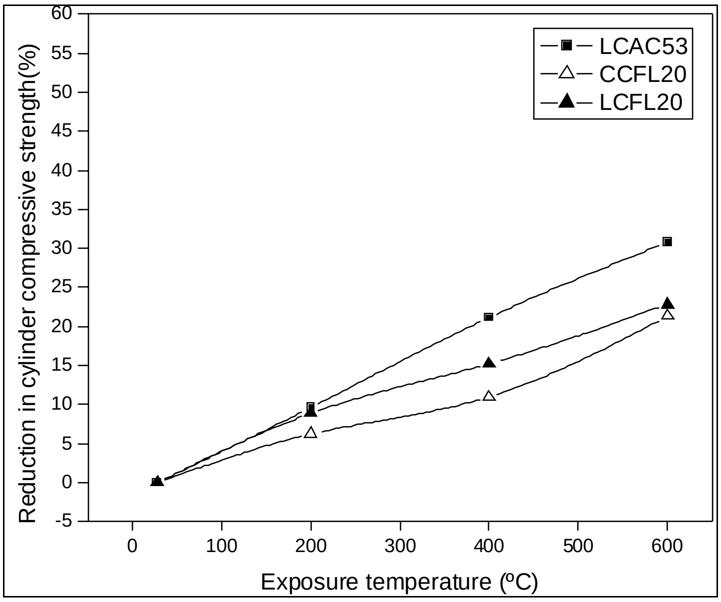

Figure 6.7 Percentage reduction in cylinder compressive strength of concrete with temperature after air cooling.

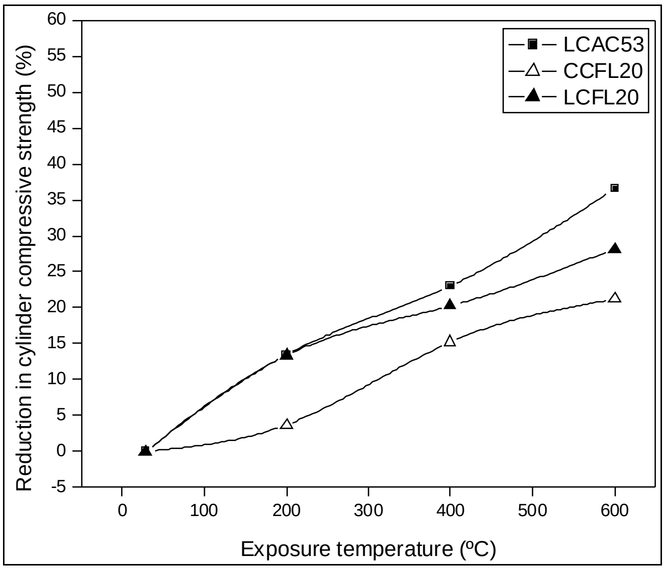

Figure 6.8 Percentage reduction in cylinder compressive strength of concrete with temperature after water cooling.

From Figures 6.5 to 6.8, it could be seen that, the reduction in strength of concrete is more or less the same up to 200ºC and beyond this, the strength reduction is low in both CC and LC when fly ash is added. A similar behaviour was observed by Poon et al. [4]. In the present study cube compressive strength of CCFL20 and LCFL20 was reduced by 19% and 17% at 600ºC under air cooling and the corresponding reduction was 23% and 20% for water cooled environment. The cylinder compressive strength of CCFL20 and LCFL20 was reduced by 21% and 23% at 600ºC under air cooling and the corresponding reduction was 21% and 28% for water cooled environment. However at 600ºC control concrete had a reduction in cube strength by 29% and 32% for air and water environments and the corresponding reduction in cylinder compressive strength was 29% and 34%.

Figures 6.9 to 6.12 depict the percentage reduction in compressive strength of LCAC53, CCGG25 and LCGG25 at elevated temperature under both environments compared to the ambient compressive strength of respective concrete.

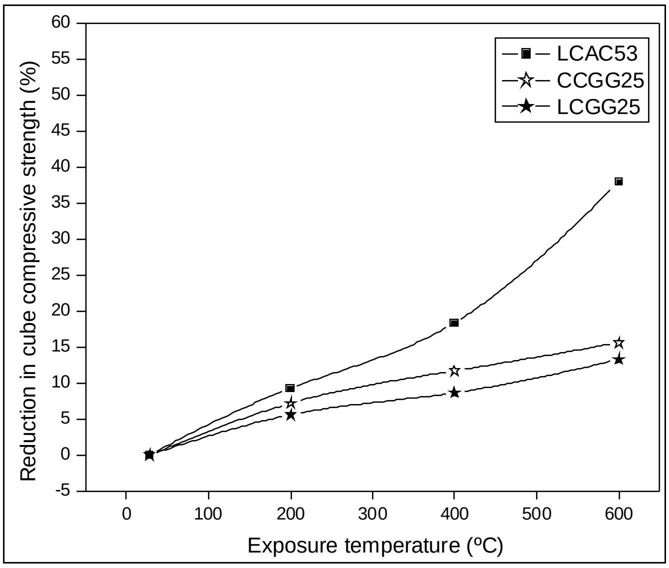

Figure 6.9 Percentage reduction in cube compressive strength concrete with temperature after air cooling.

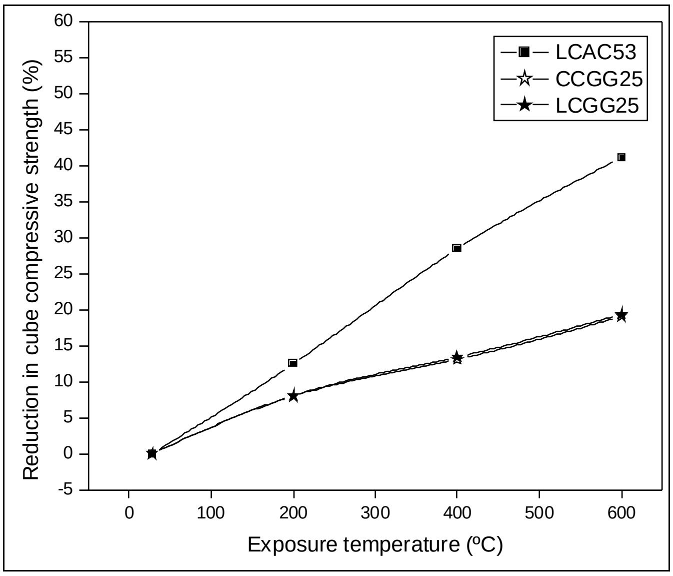

Figure 6.10 Percentage reduction in cube compressive strength of concrete with temperature after water cooling.

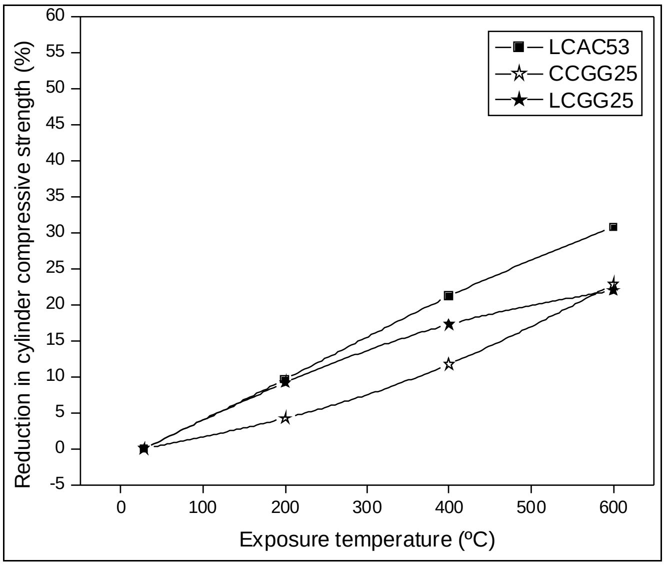

Figure 6.11 Percentage reduction in cylinder compressive strength of concrete with temperature after air cooling.

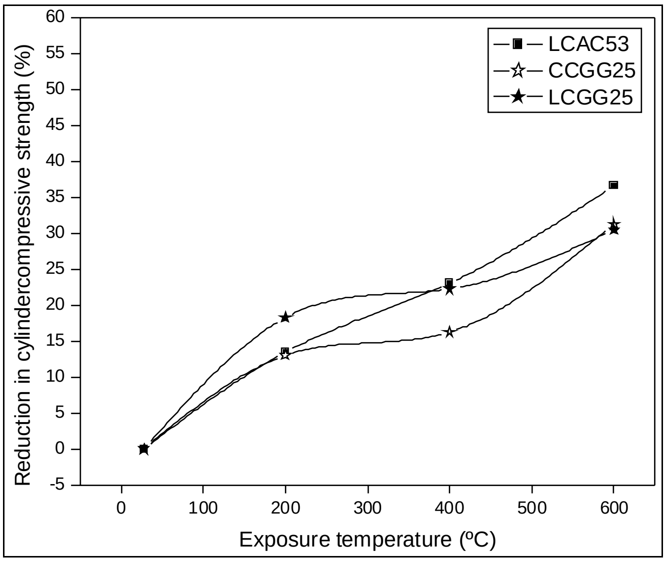

Figure 6.12 Percentage reduction in cylinder compressive strength of concrete with temperature after water cooling.

From Figures 6.9 to 6.12, it can be observed that the reduction in compressive strength with GGBFS is almost similar to concrete with fly ash as supplementary cementitious material. At 600ºC the percentage reduction in cube strength of CCGG25 and LCGG25 under air cooled environment was 16% and 13% and the corresponding reduction for water cooled environment was 19% and 18% respectively. The corresponding values for cylinder specimen were 23%, 22%, 36% and 35% respectively.

In general, it could be concluded that supplementary cementitious materials (mineral admixtures) improve the residual compressive strength significantly after the exposure to high temperature. In the present study while laterised concrete with OPC alone gave a strength reduction in the range of 30% to 50% at 600ºC, the concrete with supplementary cementitious materials (both fly ash and GGBFS) showed a strength reduction in the range of 10% to 32% only at 600ºC. This behavior is attributed due to the fact that supplementary cementitious materials increase resistance against micro cracking.

Figures 6.13 and 6.14 depict the percentage reduction in compressive strength of LSCCF and LSCCG at elevated temperature under both environments compared to the ambient compressive strength of respective concrete.

From Figures 6.13 and 6.14, as well as from Tables 6.1 and 6.2, it could be observed that, laterised self compacting concrete with GGBFS as addition (LSCCG) shows higher strength at ambient temperature when compared with corresponding laterised self compacting concrete with fly ash as addition (LSCCF). However, with higher temperature, the loss of strength in LSCCG is higher compared to LSCCF. In the present study LSCCG gave 8.5% higher cube compressive strength at ambient temperature compared to LSCCF. However at 600ºC, compared to the strength of LSCCF, LSCCG got 10.5% and 15% less strength respectively for air cooled and water cooled environment. Cylinder strength also shows a similar trend. It has been reported elsewhere

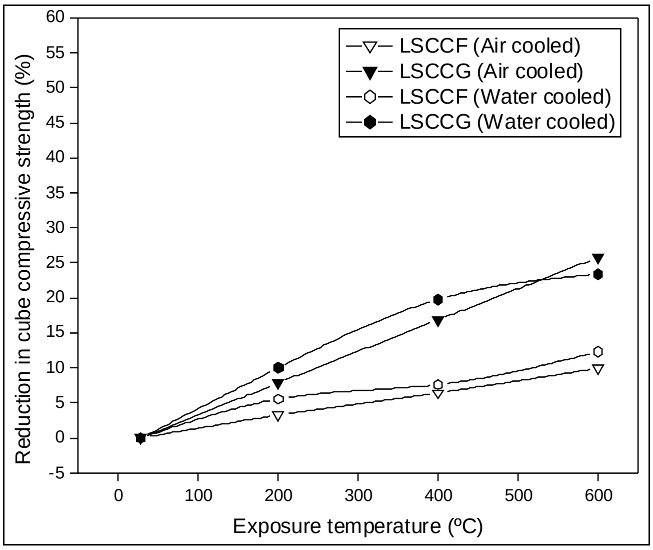

Figure 6.13 Percentage reduction in cube compressive strength of self compacting concrete with temperature after cooling under different environments.

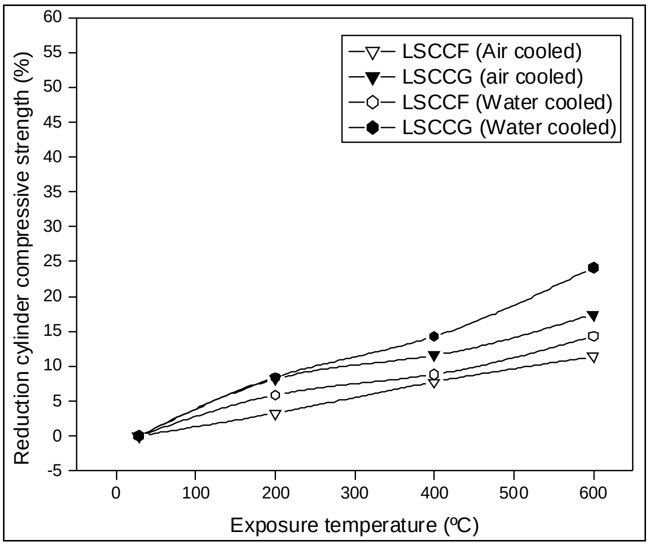

Figure 6.14 Percentage reduction in cylinder compressive strength of self compacting concrete with temperature after air cooling under different environments.

[59,103] that the strength of normal concrete with GGBFS performed better compared to fly ash concrete at elevated temperature, which is contradictory to the results of present investigation. However many researchers observed [56,103] better strength for normal concrete with fly ash compared to concrete with OPC when exposed to higher temperature. The poor performance of LSCCG at high temperature could be due to the presence of large quantity of GGBFS in LSCCG.

The Figures 6.15 and 6.16 compare the variation of cube compressive strength between self compacting laterised concrete and vibrated laterised concrete for air cooled and water cooled environment.

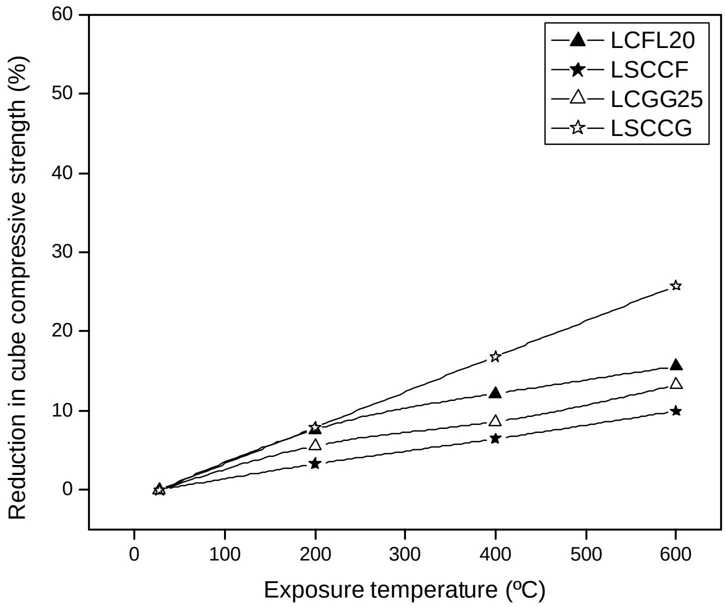

Figure 6.15 Percentage reduction in cube compressive strength of concrete with temperature after air cooling.

From these figures, it could be seen that, compared to vibrated concrete, laterised self compacting concrete with fly ash as addition shows less percentage reduction in cube compressive strength with increase in temperature.

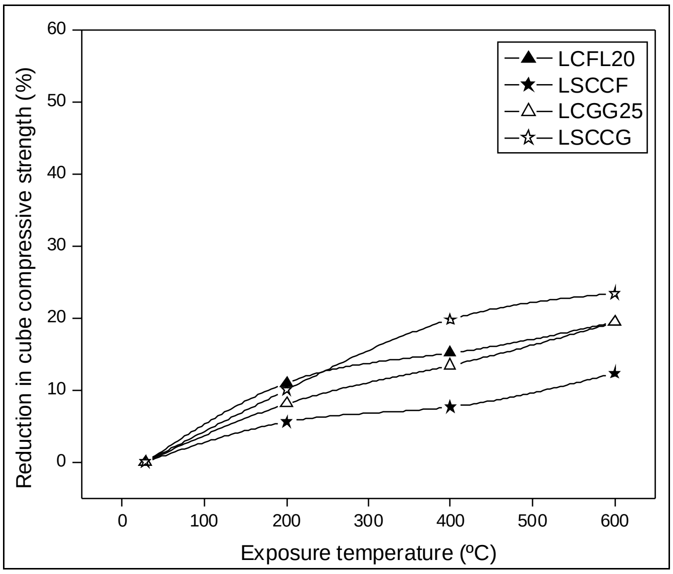

Figure 6.16 Percentage reduction in cube compressive strength of concrete with temperature after water cooling.

The Figure 6.17 and 6.18 compare the variation of cylinder compressive strength between laterised self compacting concrete and vibrated laterised concrete for air cooled and water cooled environment.

From these figures, it could be seen that, laterised self compacting concrete with fly ash as addition shows less percentage reduction in cylinder compressive strength with increase in temperature compared to vibrated concrete.

Different investigators have proposed expression for predicting the cube compressive strength of conventional concrete at elevated temperatures [62, 63, 60,119,120,121, and 122].

The predicted cube compressive strength of CC, LCF, LCAC53, CCFL20, LCFL20, CCGG25 and LCGG25 based on the equations for conventional concrete are compared with corresponding experimental values under both environment (air cooled and water

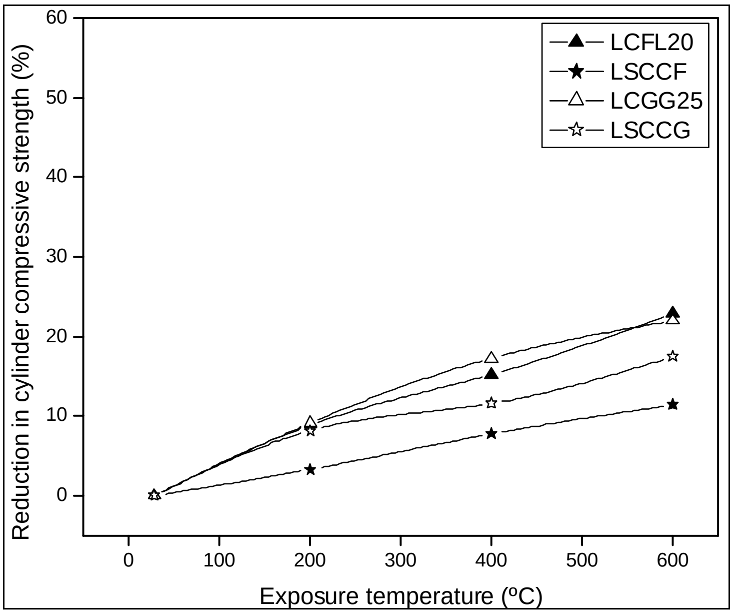

Figure 6.17 Percentage reduction in cylinder compressive strength of concrete with temperature after air cooling.

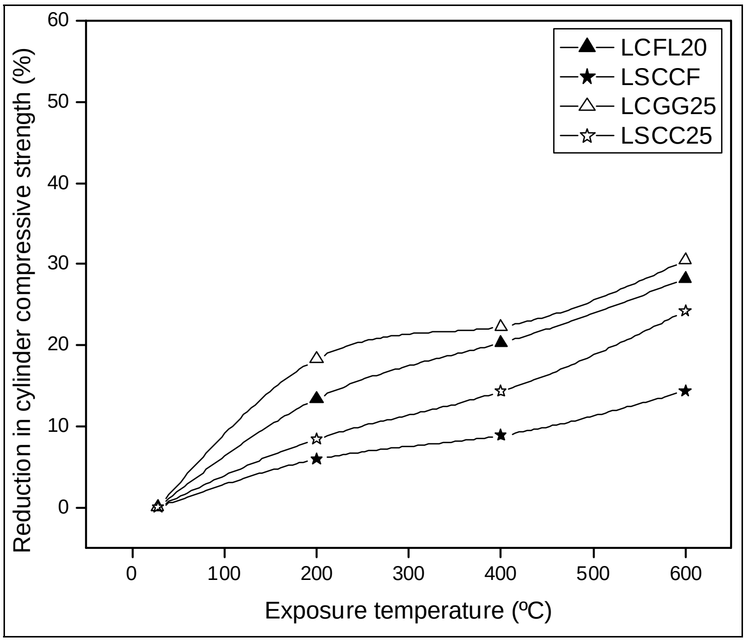

Figure 6.18 Percentage reduction in cylinder compressive strength of concrete with temperature after water cooling.

cooled) in Figures 6.19 to 6.25. Various equations used to compare the experimental results are presented in section 2.7 of Chapter 2.

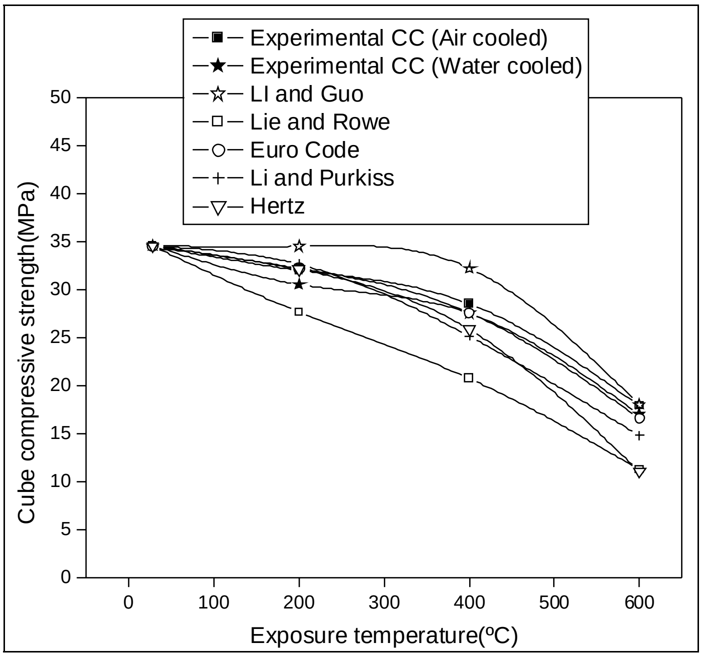

Figure 6.19 Cube compressive strength-temperature relationship of CC

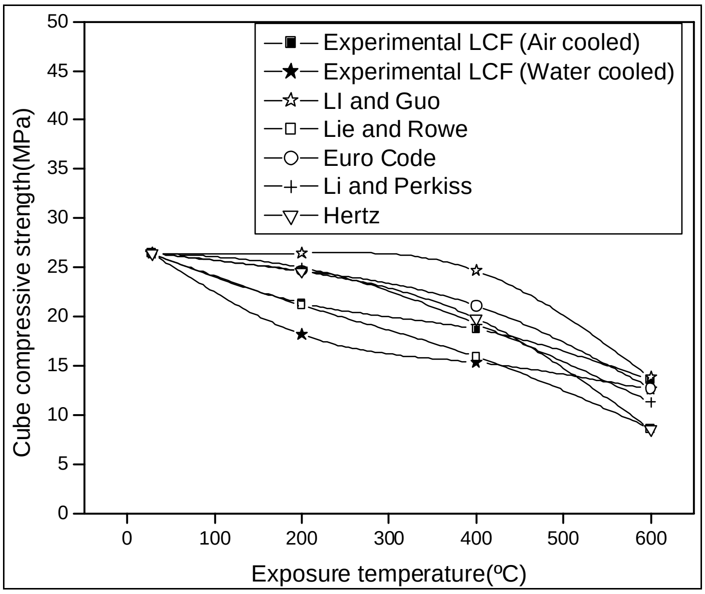

Figure 6.20 Cube compressive strength-temperature relationship of LCF

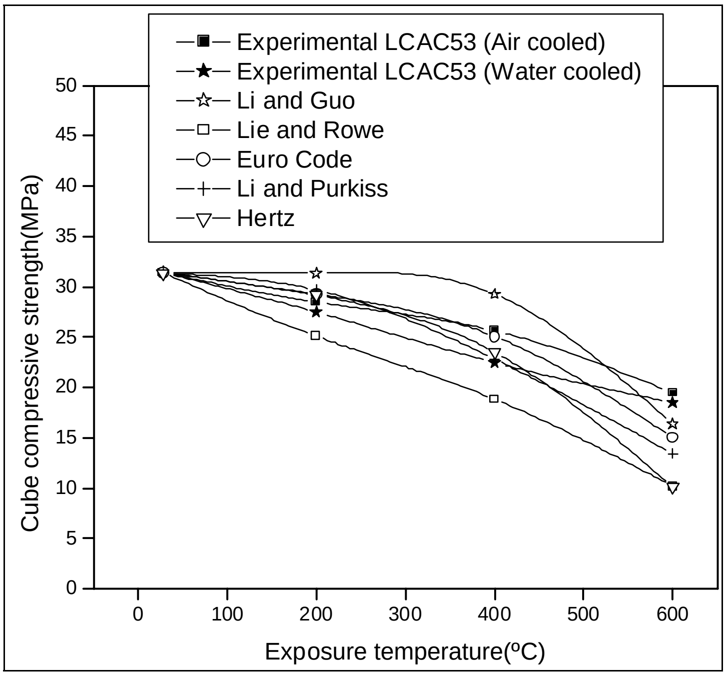

Figure 6.21 Cube compressive strength-temperature relationship of LCAC53

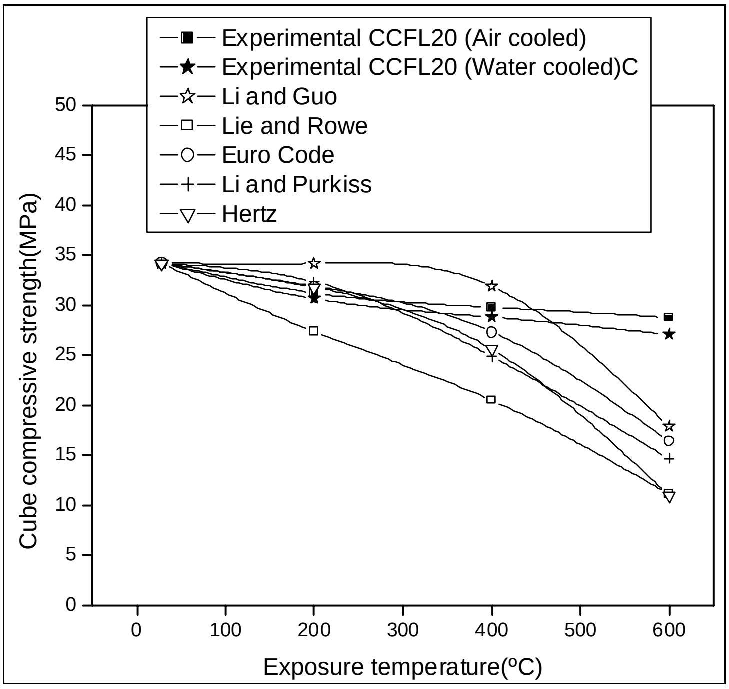

Figure 6.22 Cube compressive strength-temperature relationship of CCFL20

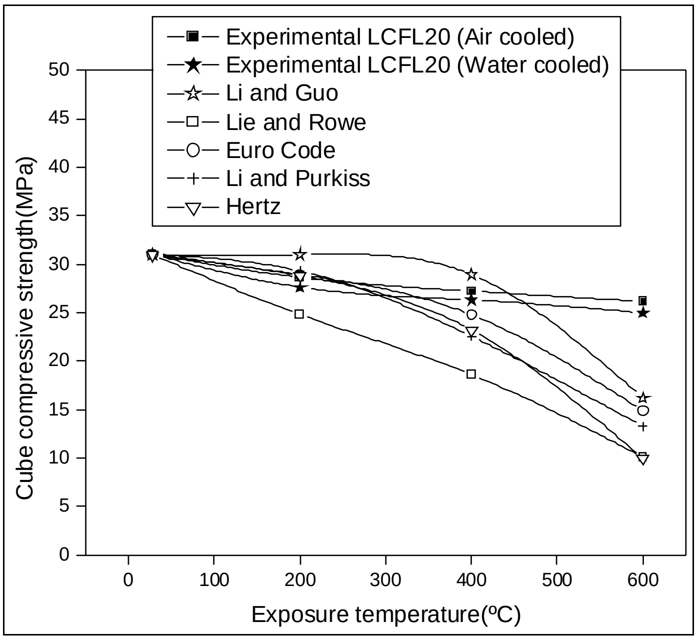

Figure 6.23 Cube compressive strength-temperature relationship of LCFL20

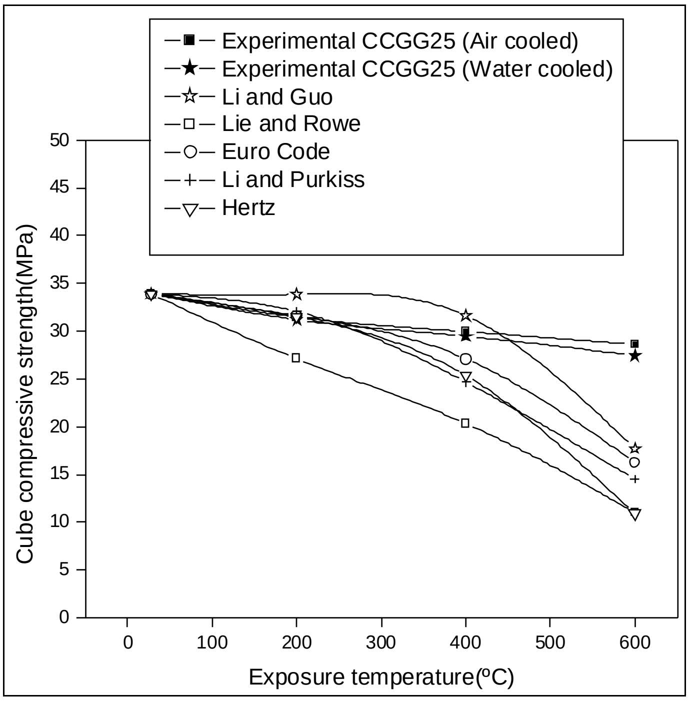

Figure 6.24 Cube compressive strength-temperature relationship of CCGG25

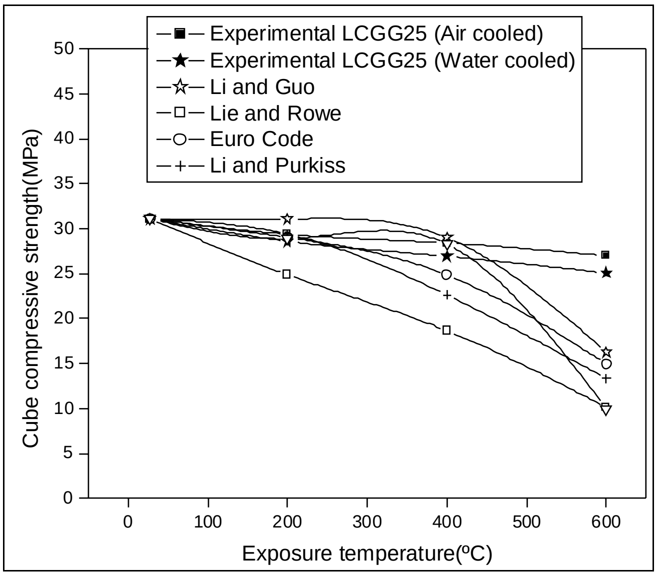

Figure 6.25 Cube compressive strength-temperature relationship of LCGG25

From Figures 6.19 to 6.25, it could be seen that, among various investigators there is a wide range of variation in the prediction of cube compressive strength with temperature. Available prediction equations underestimate the strength of laterised concrete beyond 500ºC. Further, while available equations predict a rapid strength reduction beyond 400ºC, the experimental results show that when fly ash or GGBFS is used as addition in concrete, the rate of strength reduction is almost linear up to 600ºC and that there is no significant strength reduction at 600ºC.

It could be concluded that a new equation is required for the prediction of cube strength of laterised concrete with temperature when fly ash or GGBFS is added in concrete. The proposed equation based on the present experimental study is a lower bound prediction of cube compressive strength of laterised concrete with temperature when supplementary cementitious materials are added (fly ash or GGBFS) and is presented in equation 6.1. This equation has been proposed based on the strength test of M25 grade concrete and further study is needed to generalise the proposed equation.

Figure 6.26 shows the scatter diagram of the cube compressive strength of laterised concrete modified with supplementary cementitious materials when exposed to elevated temperature.

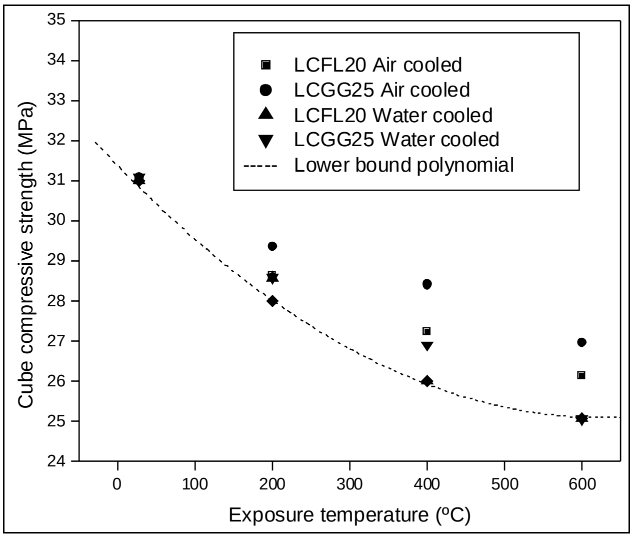

Figure 6.26 Scatter diagram of the cube compressive strength of laterised concrete modified with supplementary cementitious materials when exposed to elevated temperature.

$f_{cT} = f_{ck}[1.013\text{-}0.65(T/1000)\text{-}0.51(T/1000)^2]$ for 28< T >600 ....(6.1)

6.3 Tensile Strength

The flexural and cylinder split tensile strength results for various types of concrete are presented in Tables 6.3 and 6.4 respectively.

From Table 6.4, it could be seen that, at ambient temperature, the control concrete shows higher split tensile strength compared to all other types of concrete. When fly ash or GGBFS is added in concrete, the rate of strength reduction with temperature is less compared to CC and this rate of reduction is further reduced when laterite aggregates along with fly ash/GGBFS is used in concrete.

Table 6.3 Flexural strength of concrete after the exposure to elevated temperature

Sl. No.	Concrete type (M25 grade)	Flexural strength (MPa)													
		Ambient 28°C		Air cooled						Water cooled					
				Exposure temperature(°C)						Exposure temperature(°C)					
				200		400		600		200		400		600	
		Mean	SD	Mean	SD	Mean	SD	Mean	SD	Mean	SD	Mean	SD	Mean	SD
1	CC	5.47	0.56	5.00	0.38	3.99	0.55	3.02	0.14	4.45	0.28	3.73	0.20	2.75	0.20
2	LCF	5.38	0.53	5.13	0.28	4.19	0.18	3.52	0.28	4.81	0.16	3.93	0.26	3.02	0.13
3	LCAC53	4.20	0.29	4.05	0.11	3.37	0.20	2.88	0.38	3.95	0.55	3.21	0.32	2.63	0.24
4	CCFL20	4.88	0.55	4.67	0.52	3.33	0.24	2.75	0.16	3.84	0.14	3.23	0.29	2.63	0.24
5	LCFL20	4.29	0.17	3.65	0.28	2.61	0.23	2.12	0.15	3.32	0.19	2.59	0.19	1.97	0.26
6	CCGG25	5.16	0.59	4.47	0.06	3.48	0.34	2.50	0.28	3.61	0.17	3.03	0.22	2.21	0.16
7	LCGG25	4.48	0.11	3.73	0.15	2.75	0.20	1.95	0.13	3.28	0.18	2.43	0.22	1.72	0.21
8	LSCCF	5.30	0.30	4.30	0.18	3.80	0.36	3.50	0.33	4.00	0.32	3.40	0.44	3.30	0.23
9	LSCCG	5.23	0.44	4.58	0.32	3.99	0.43	3.28	0.17	4.38	0.30	3.87	0.21	3.22	0.21

Table 6.4 Cylinder split tensile strength of concrete after the exposure to elevated temperature

Sl. No.	Concrete type (M25 grade)	Cylinder split tensile strength (MPa)													
		Ambient 28°C		Air cooled						Water cooled					
				Exposure temperature(°C)						Exposure temperature(°C)					
				200°C		400°C		600°C		200°C		400°C		600°C	
		Mean	SD	Mean	SD	Mean	SD	Mean	SD	Mean	SD	Mean	SD	Mean	SD
1	CC	4.36	0.05	3.40	0.34	2.68	0.22	2.16	0.10	3.35	0.40	2.61	0.25	2.06	0.13
2	LCF	2.24	0.33	2.05	0.18	1.70	0.09	1.40	0.13	1.90	0.26	1.53	0.09	1.25	0.11
3	LCAC53	2.93	0.42	2.80	0.23	2.32	0.11	2.10	0.29	2.78	0.26	2.24	0.09	2.00	0.27
4	CCFL20	3.77	0.33	3.30	0.38	2.81	0.19	2.41	0.14	3.20	0.33	2.57	0.27	1.89	0.19
5	LCFL20	2.90	0.17	2.55	0.04	2.34	0.13	2.13	0.26	2.50	0.19	2.12	0.26	1.65	0.13
6	CCGG25	3.99	0.50	3.52	0.42	2.99	0.03	2.49	0.16	3.15	0.31	2.87	0.14	1.79	0.12
7	LCGG25	3.24	0.16	3.04	0.44	2.62	0.28	2.15	0.07	2.63	0.34	2.34	0.16	1.67	0.13
8	LSCCF	3.28	0.27	2.76	0.17	2.40	0.27	2.20	0.07	2.60	0.07	2.30	0.27	2.10	0.30
9	LSCCG	3.65	0.31	3.12	0.27	2.33	0.20	1.98	0.30	3.01	0.08	2.42	0.16	1.90	0.06

The tensile strength reduction of CC, LCF and LCAC53 with temperature is compared with their respective strength at ambient temperature in Figures 6.27 to 6.30. Figures 6.27 and 6.28 depicts the percentage reduction in split tensile strength with temperature under air cooled and water cooled environment respectively. The corresponding comparison in the case of flexural strength is shown in Figures 6.29 and 6.30.

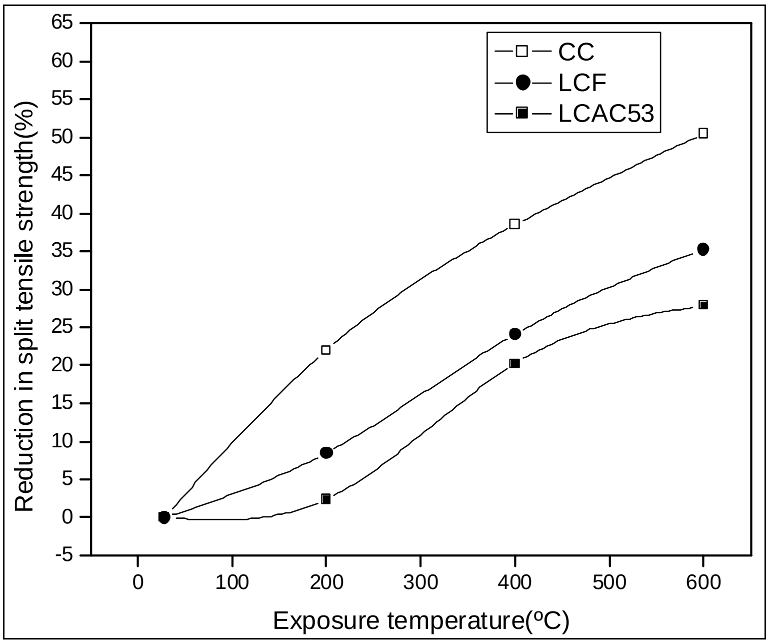

Figure 6.27 Percentage reduction in split tensile strength of concrete with temperature after air cooling.

When sand is replaced with laterite fine aggregate in concrete, the rate of reduction in split tensile strength of concrete reduces. The rate of reduction is still less when entire aggregate in concrete are replaced with laterite all-in aggregate which is irrespective of cooling environment. Similar behaviour could be seen in case of flexural strength also.

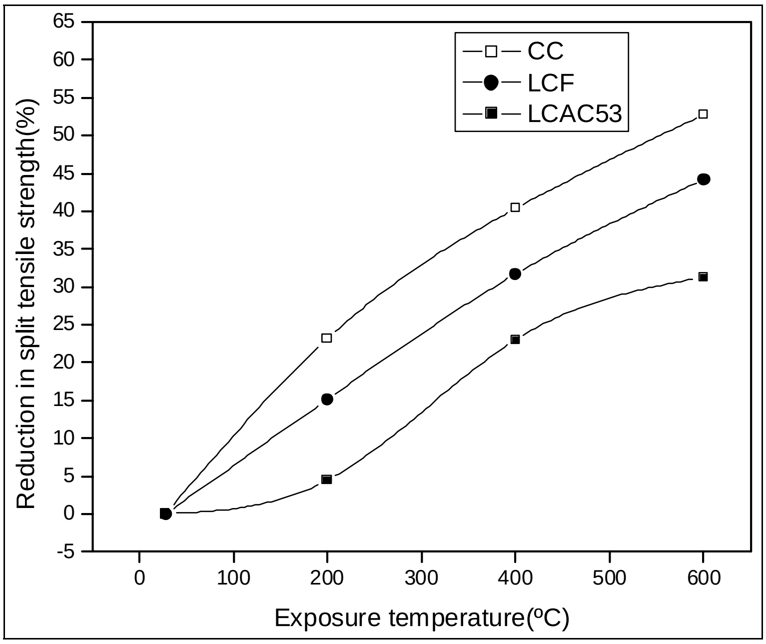

Figure 6.28 Percentage reduction in split tensile strength of concrete with temperature after water cooling.

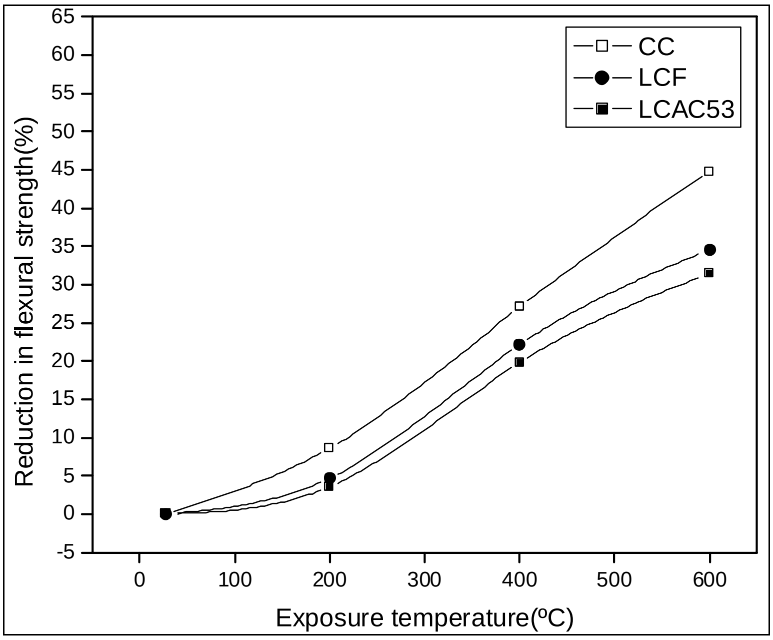

Figure 6.29 Percentage reduction in flexural strength of concrete with temperature after air cooling.

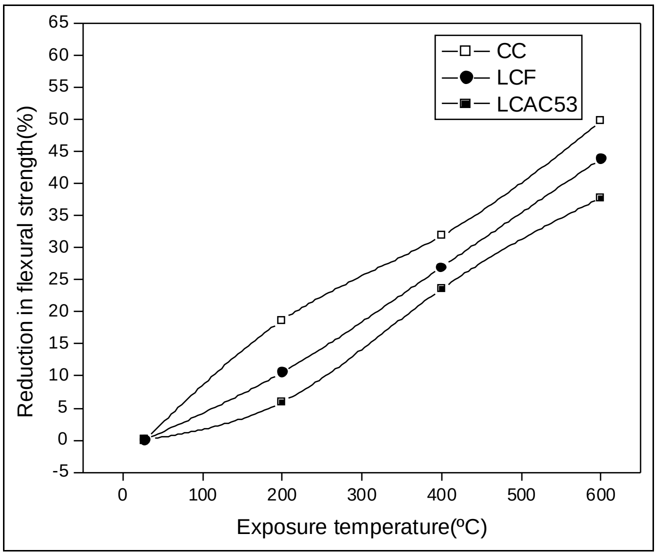

Figure 6.30 Percentage reduction in flexural strength of concrete with temperature after water cooling.

Figurers 6.31 to 6.34 depict the percentage reduction in tensile strength of LCAC53, CCFL20 and LCFL20 at elevated temperature.

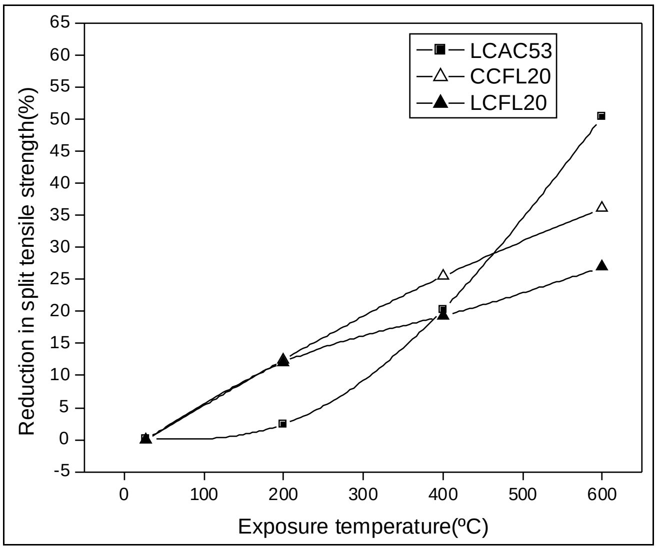

Figure 6.31 Percentage reduction in split tensile strength of concrete with temperature after air cooling.

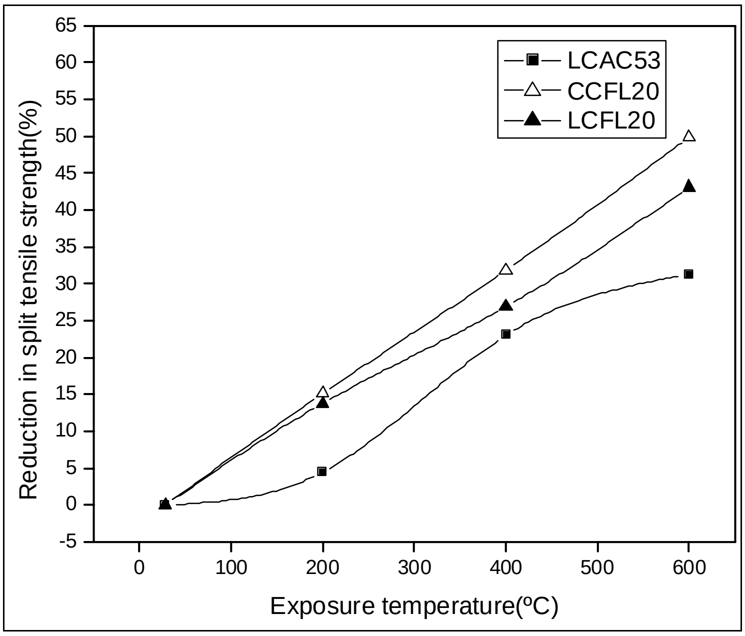

Figure 6.32 Percentage reduction in split tensile strength of concrete with temperature after water cooling.

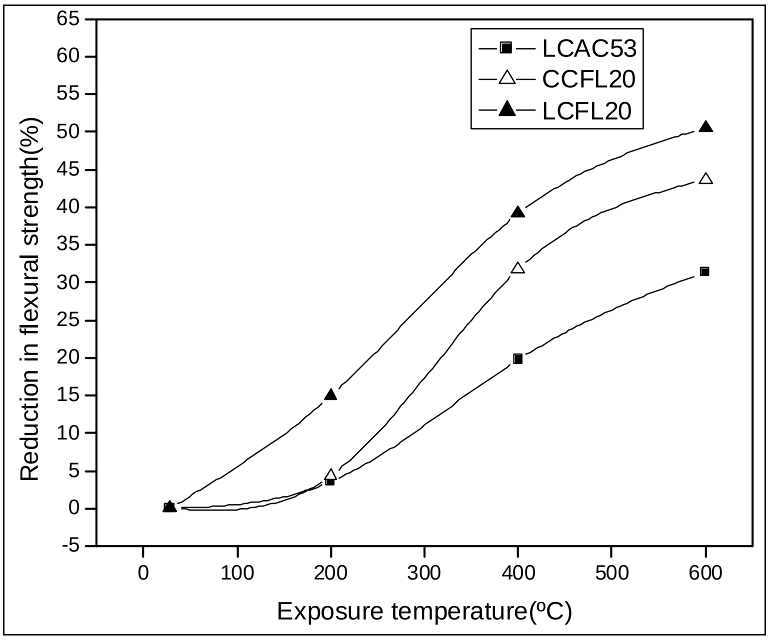

Figure 6.33 Percentage reduction in flexural strength of concrete with temperature after air cooling.

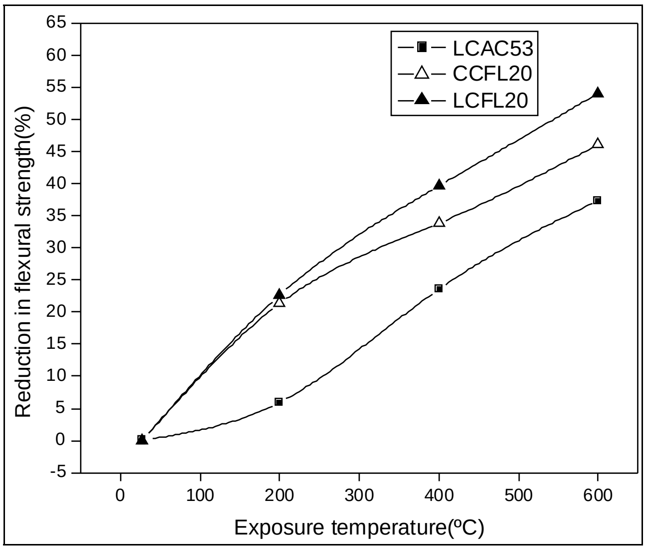

Figure 6.34 Percentage reduction in flexural strength of concrete with temperature after water cooling.

From Figures 6.31 to 6.34, it could be seen that, the reduction in tensile strength of concrete is more or less the same up to 200ºC and beyond this; the strength reduction is low when fly ash is added in both CC and LC. In the present study, the flexural strength of CCFL20 and LCFL20 was reduced by 44% and 51% at 600ºC under air cooling and the corresponding reduction was 46% and 54% for water cooled environment. The split tensile strength of CCFL20 and LCFL20 was reduced by 36% and 27% at 600ºC under air cooling and the corresponding reduction was 50% and 43% for water cooled environment. It may be noted that, at 600ºC control concrete had a reduction in split tensile strength by 50% and 53% for air and water environments and the corresponding reduction in flexural strength was 45% and 50%.

Figures 6.35 to 6.38 depict the percentage reduction in tensile strength of LCAC53, CCGG25 and LCGG25 at elevated temperature under both environments.

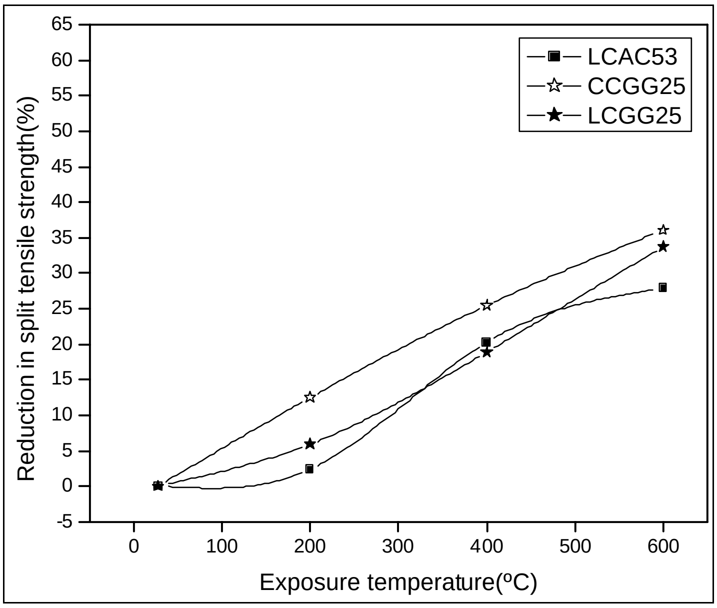

Figure 6.35 Percentage reduction in split tensile strength of concrete with temperature after air cooling.

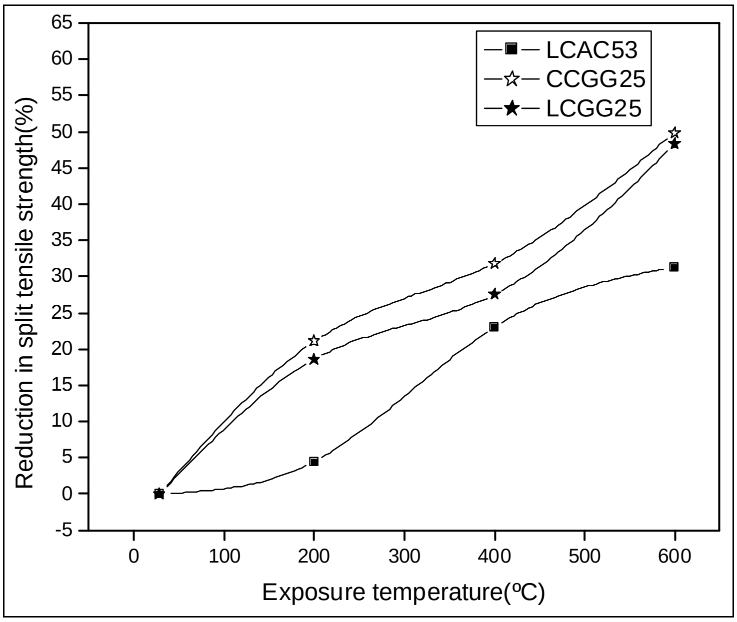

Figure 6.36 Percentage reduction in split tensile strength of concrete with temperature after water cooling.

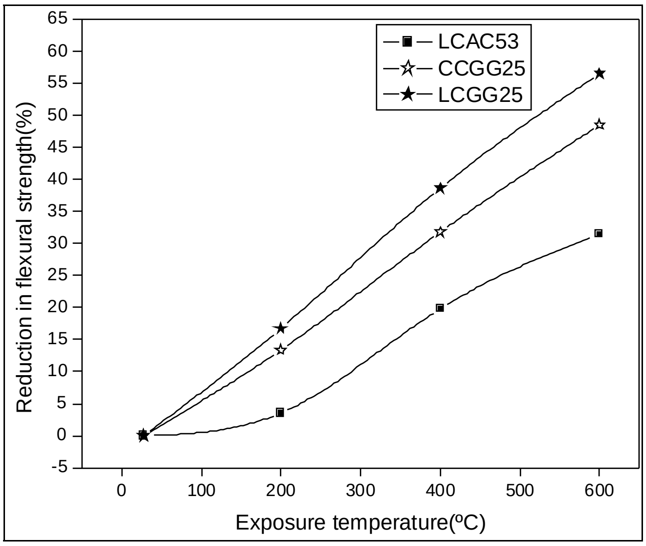

Figure 6.37 Percentage reduction in flexural strength of concrete with temperature after air cooling.

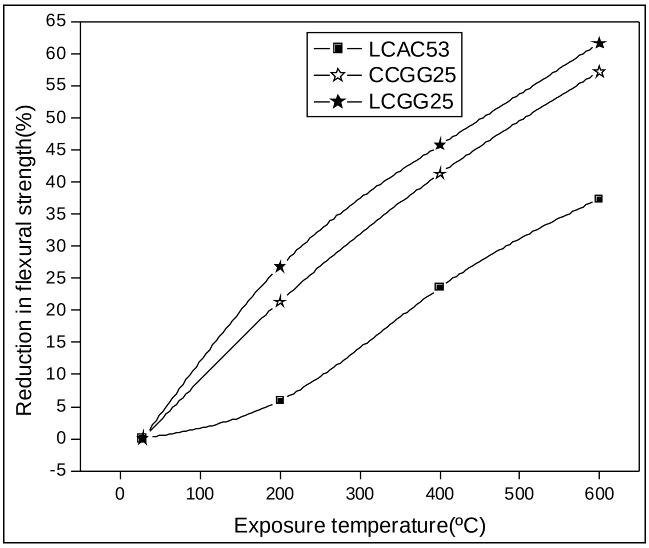

Figure 6.38 Percentage reduction in flexural strength of concrete with temperature after water cooling.

It can be observed that the reduction in split tensile strength of concrete with GGBFS is almost similar to concrete with fly ash as supplementary cementitious material. At 600ºC the percentage reduction in cylinder split strength of CCGG25 and LCGG25 under air cooled environment was 38% and 34% and the corresponding reduction for water cooled environment was 55% and 48% respectively. The corresponding values for flexure specimen were 52%, 56%, 57% and 62% respectively.

Figures 6.39 and 6.40 depict the percentage reduction in tensile strength of LSCCF and LSCCG at elevated temperature under both environments.

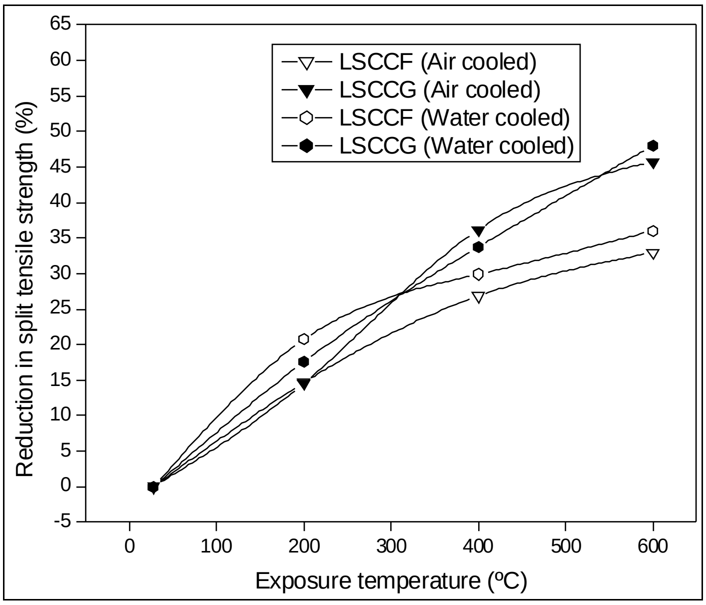

Figure 6.39 Percentage reduction in split tensile strength of laterised self compacting concrete with temperature under different cooling environment.

From Tables 6.3 and 6.4 and Figures 6.39 and 6.40, it could be seen that, even though LSCC with GGBFS shows higher split tensile strength at ambient temperature, with increase in exposure temperature, LSCC with GGBFS experiences a higher rate of strength reduction when compared to LSCC with fly ash. This may be attributed to inhomogeneity in distribution of hydration product in slag concrete at elevated temperature when compared to fly ash concrete [123].

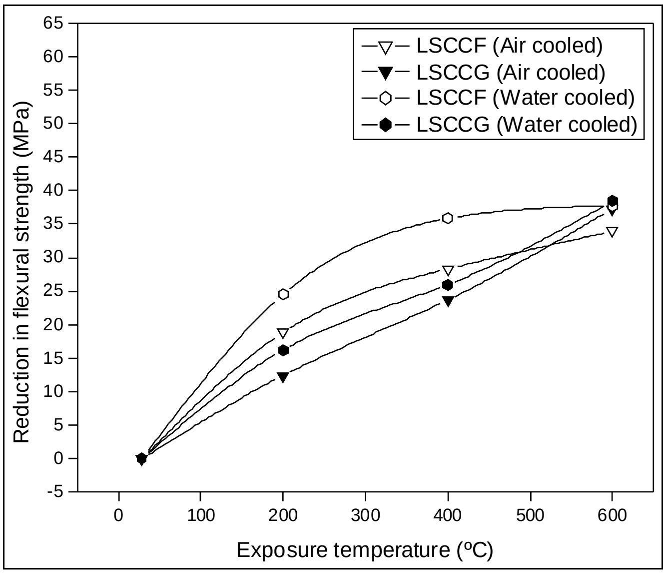

Figure 6.40 Percentage reduction in flexural strength of laterised self compacting concrete with temperature under different cooling environment

Figures 6.41 and 6.42 compare the variation of split tensile strength between self compacting laterised concrete and vibrated laterised concrete respectively for air cooled and water cooled environment.

From these figures, it could be seen that, laterised self compacting concrete with both fly ash and GGBFS as additions show higher percentage reduction in split tensile strength with increase in temperature compared to vibrated concrete.

Figures 6.43 and 6.44 compare the variation of flexural strength between self compacting laterised concrete and vibrated laterised concrete respectively for air cooled and water cooled environment.

Figures 6.43 and 6.44 observe that, under both air cooled and water cooled environments, the self compacting concrete with fly ash as well as GGBFS as additions have low percentage reduction in flexural strength at higher temperature levels than vibrated concrete.

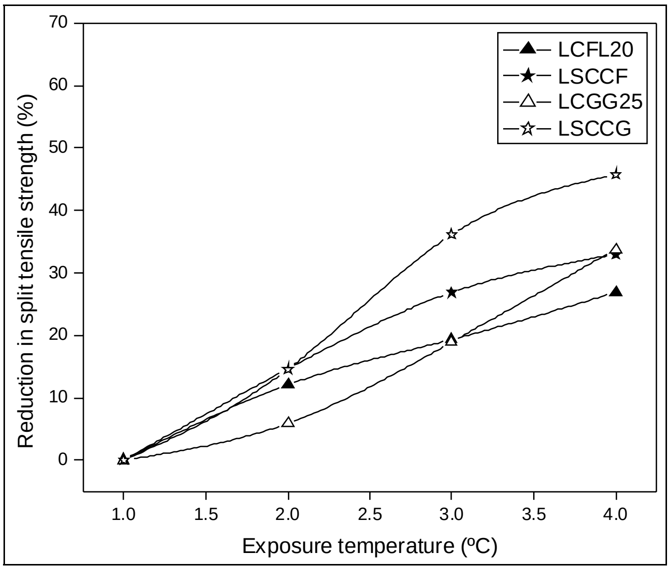

Figure 6.41 Percentage reduction in split tensile strength of concrete with temperature after air cooling.

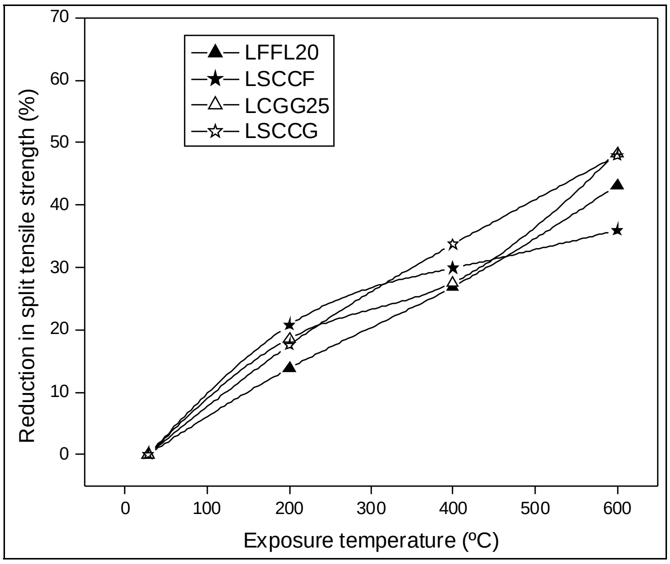

Figure 6.42 Percentage reduction in split tensile strength of concrete with temperature after water cooling.

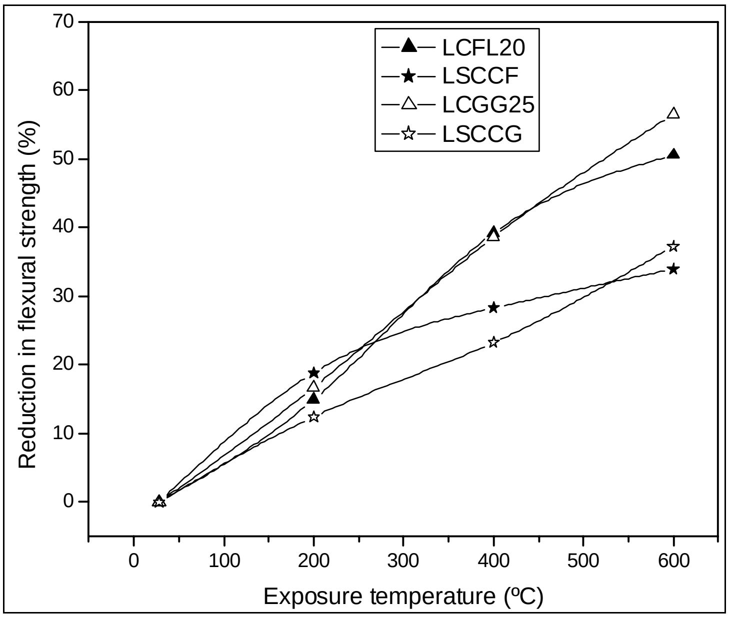

Figure 6.43 Percentage reduction in flexural strength strength of concrete with temperature after air cooling.

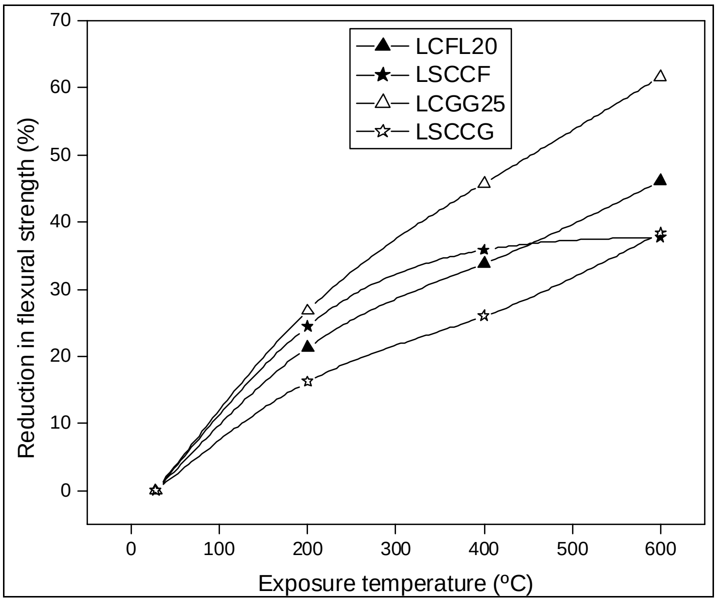

Figure 6.44 Percentage reduction in flexural strength of concrete with temperature after water cooling.

Different investigators have proposed expressions for predicting split tensile strength of conventional concrete at higher temperature based on the value at ambient temperature [60, 62, 63, 119, 120, 121, and 122]. Section 2.7 of Chapter 2 presents various mathematical models used in the prediction of cylinder split strength. Figures 6.45 to 6.51 compares the split tensile strength of concrete predicted based on different investigators with the corresponding experimental values.

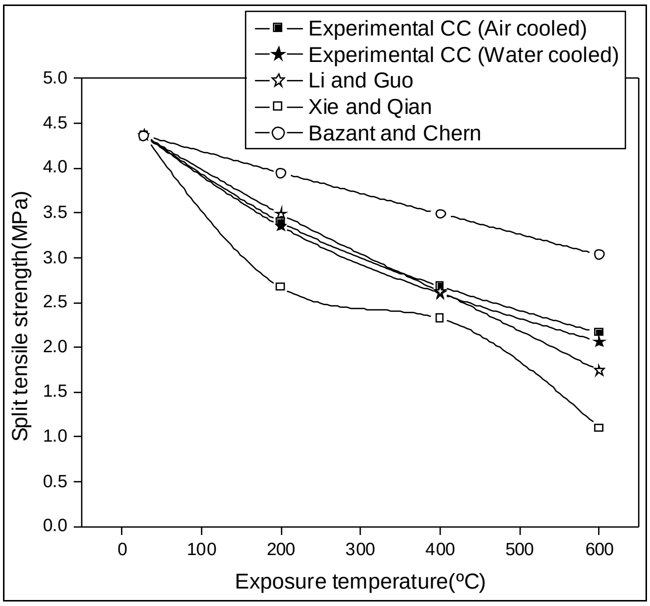

Figure 6.45 Split tensile strength-temperature relationship of CC.

From Figures 6.45 to 6.51, it could be observed that the model suggested by Bazant and Chern [60] predicts the split tensile strength of all types of laterised concrete close to the experimental value for air cooled environment. In the case of split tensile strength of laterised concrete with water cooled environment, while Bazant and Chern overestimates the strength, Li and Guo underestimates. The experimental value lies somewhat between the above predictions.

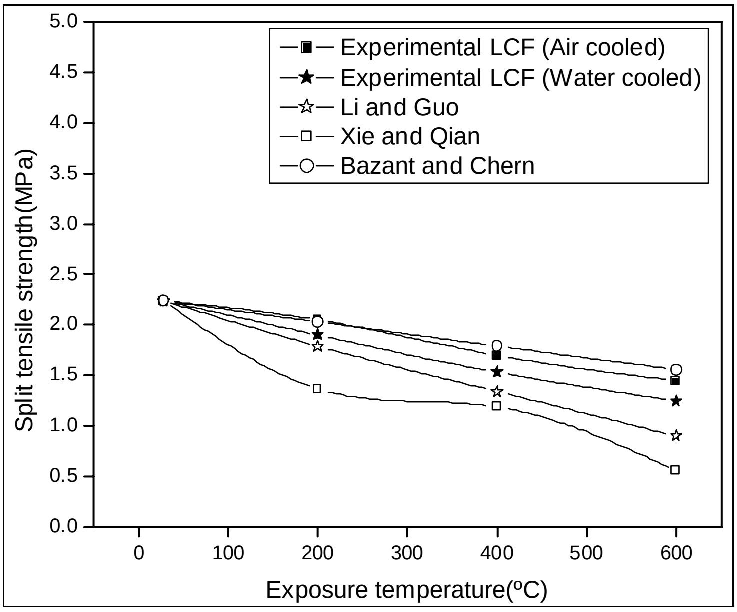

Figure 6.46 Split tensile strength- temperature relationship of LCF.

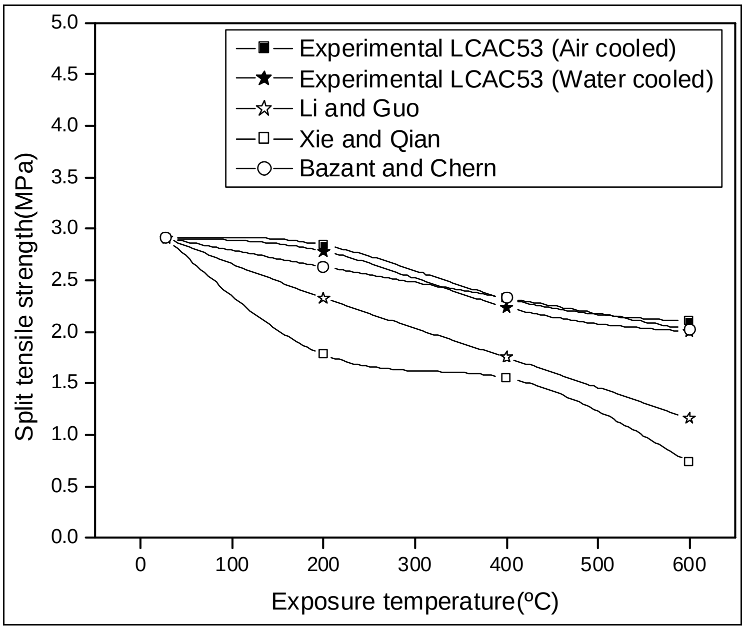

Figure 6.47 Split tensile strength- temperature relationship of LCAC53.

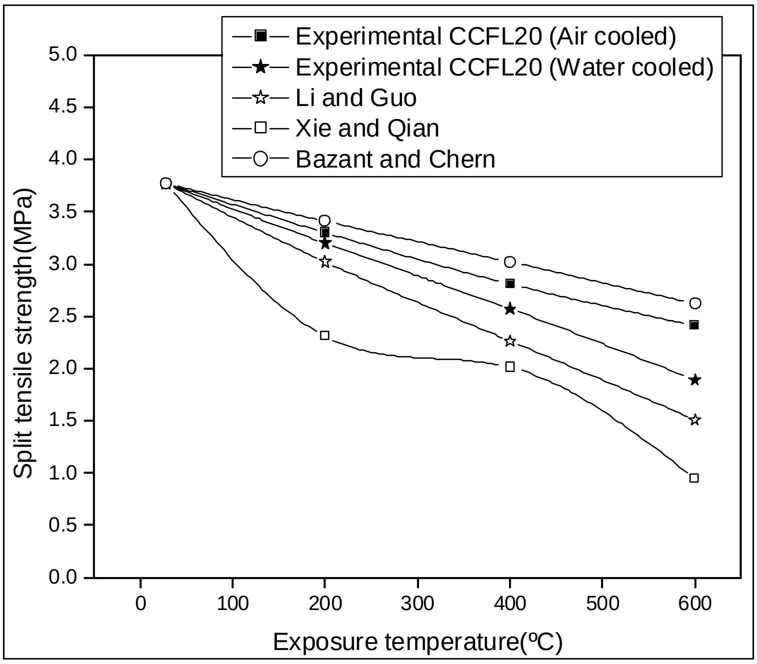

Figure 6.48 Split tensile strength-temperature relationship of CCFL20.

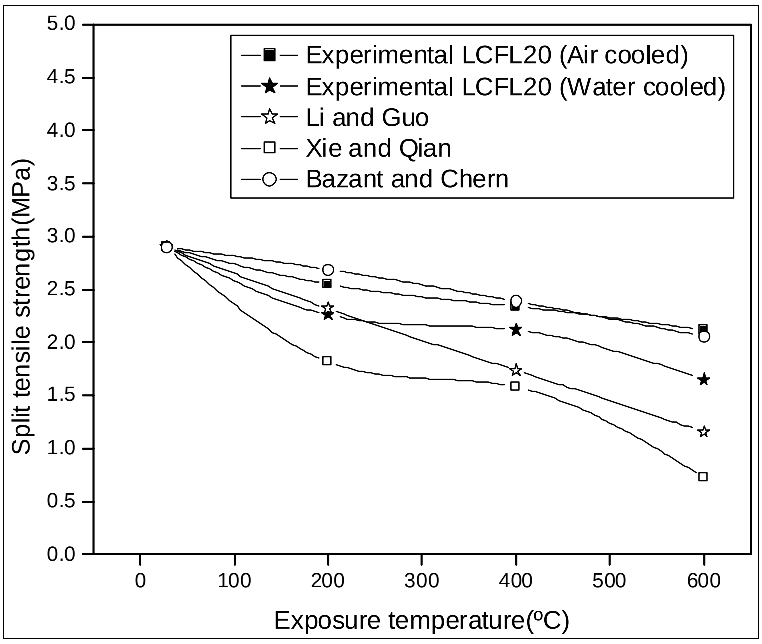

Figure 6.49 Split tensile strength-temperature relationship of LCFL20.

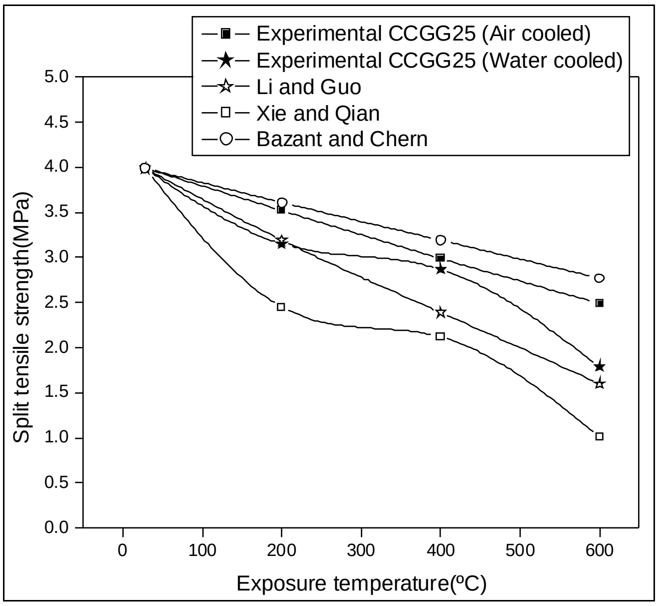

Figure 6.50 Split tensile strength-temperature relationship of CCGG25.

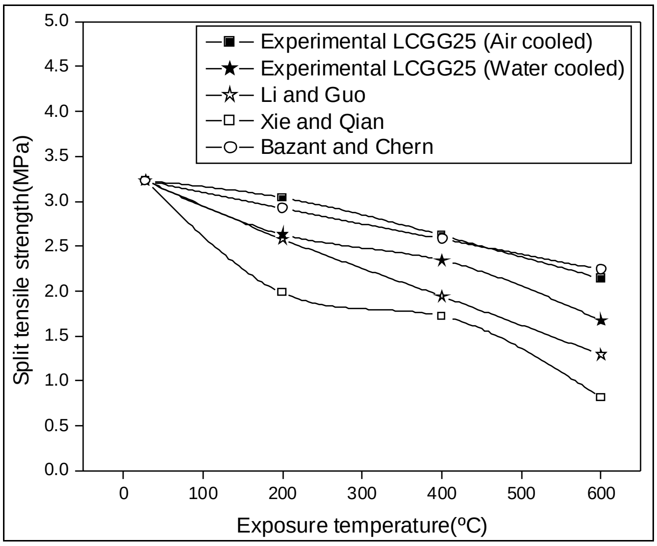

Figure 6.51 Split tensile strength-temperature relationship of LCGG25.

A lower bound equation has been proposed to predict the split tensile strength of laterised self compacting concrete with temperature and the same is presented as equations 6.2. Figure 6.52 shows the scatter-gram of the test results of split tensile strength of laterised self compacting concrete with temperature.

$$f_{TT} = f_T[\ 1.02\text{-}1.15(T/1000)\text{-}7.05(T/1000)^2\ ] \text{ for } 28< T\ >600 \text{ ....(6.2)}$$

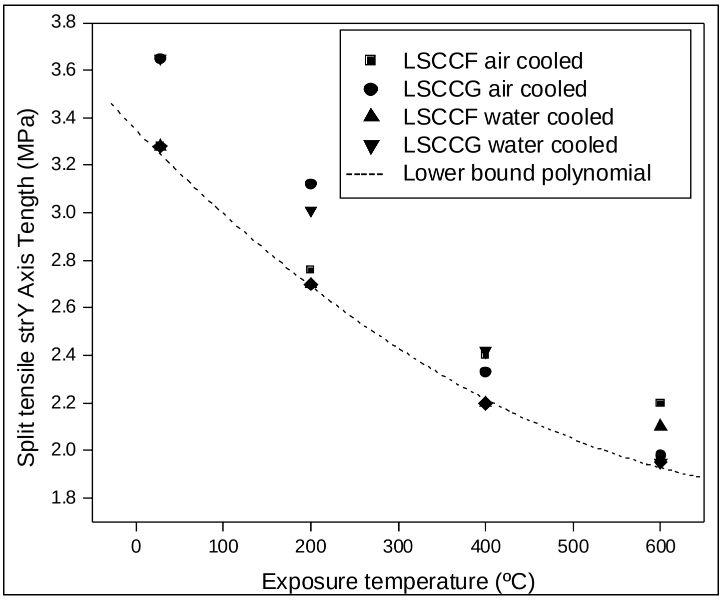

Figure 6.52 Scatter-gram of the test results of split tensile strength of laterised self compacting concrete with temperature.

6.4 Modulus of Elasticity

The modulus of elasticity of concrete calculated after exposure to different temperature levels cooled in different environments is presented in Table 6.5.

Table 6.5 shows that, irrespective of the type of concrete, the modulus of elasticity reduces with increase in temperature. Water cooled concrete have less modulus of elasticity compared to air cooled concrete. LC series of concrete showed lower modulus of elasticity compared to CC series of concrete.

Table 6.5 Modulus of elasticity of concrete after the exposure to elevated temperature

Sl. No.	Concrete type (M25 grade)	Modulus of elasticity (MPa)													
		Ambient 28°C		Air cooled						Water cooled					
				Exposure temperature(°C)						Exposure temperature(°C)					
				200		400		600		200		400		600	
		Mean	SD	Mean	SD	Mean	SD	Mean	SD	Mean	SD	Mean	SD	Mean	SD
1	CC	34968	1520	33270	1865	28625	1382	17125	2171	32498	1697	24373	1334	14793	1981
2	LCF	27502	1105	25372	1561	16900	1099	9640	1336	24392	1060	15310	1154	8780	963
3	LCAC53	28893	2196	26254	1683	18918	2462	10114	1333	25867	2170	16250	1156	8870	87
4	CCFL20	30505	1029	28294	1252	24713	1039	17201	1577	27382	2308	21955	3177	14147	1519
5	LCFL20	28200	1448	27100	1035	24100	1925	16540	1152	26120	1425	20940	3042	13275	909
6	CCGG25	31660	1323	28399	1426	24510	923	17109	2163	26648	2257	20075	1875	14452	765
7	LCGG25	29358	1353	27305	869	23955	3037	16135	2176	25520	1794	19201	1630	13038	1021
8	LSCCF	30180	901	26679	1395	19236	2044	11003	766	26076	1953	16402	1619	8972	530
9	LSCCG	31123	2252	28238	764	20735	1915	10347	1409	27100	1699	16364	1089	9132	944

Figures 6.53 to 6.58 compare the percentage reduction in modulus of elasticity of different types of concrete.

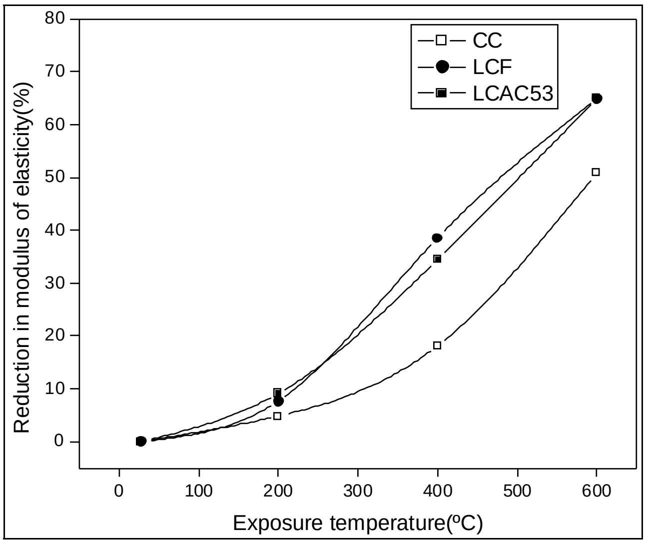

Figure 6.53 Percentage reduction in modulus of elasticity of concrete with temperature after air cooling.

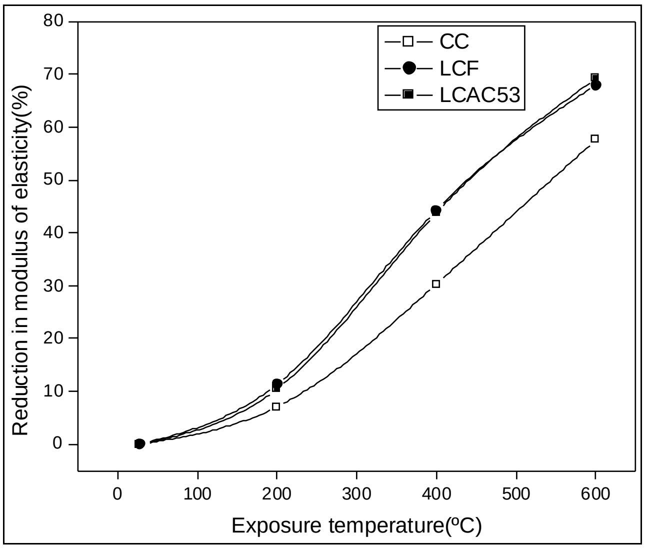

Figure 6.54 Percentage reduction in modulus of elasticity of concrete with temperature after water cooling.

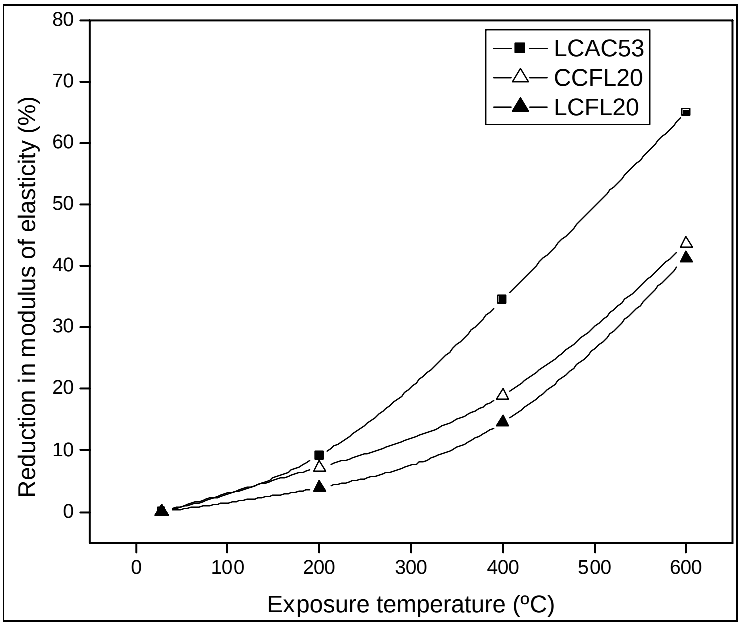

Figure 6.55 Percentage reductions in modulus of elasticity of concrete with temperature after air cooling.

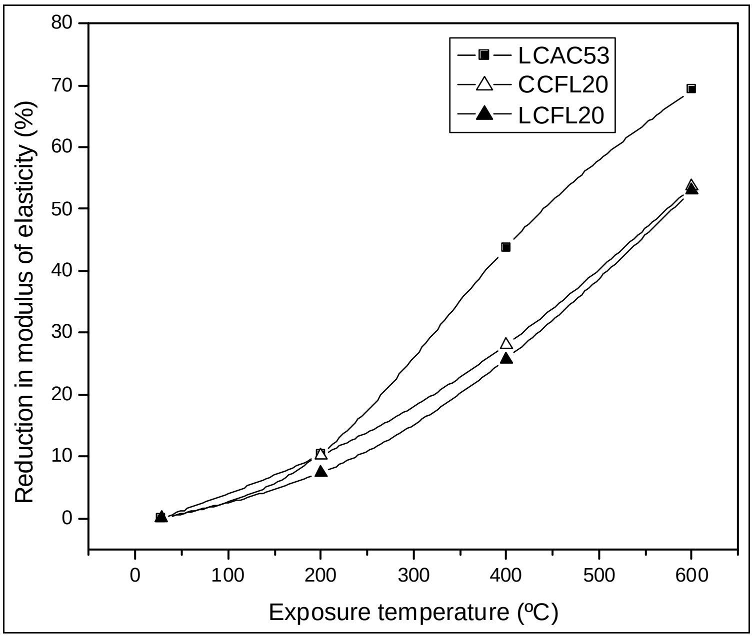

Figure 6.56 Percentage reduction in modulus of elasticity of concrete with temperature after water cooling.

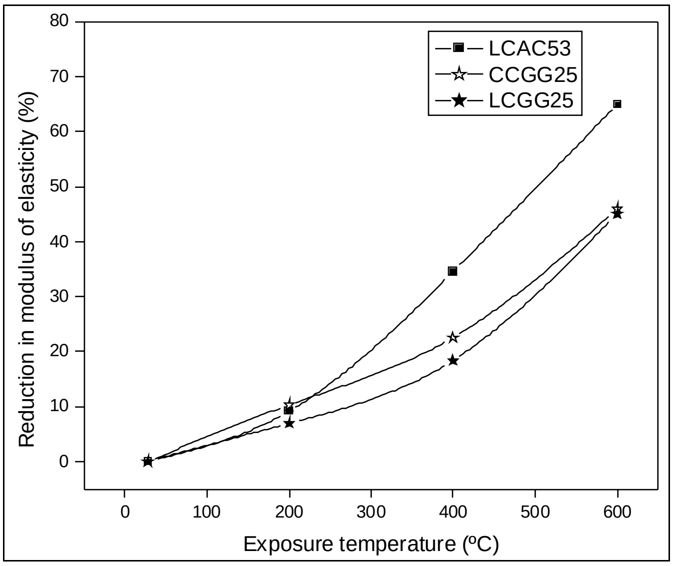

Figure 6.57 Percentage reduction in modulus of elasticity of concrete with temperature after air cooling.

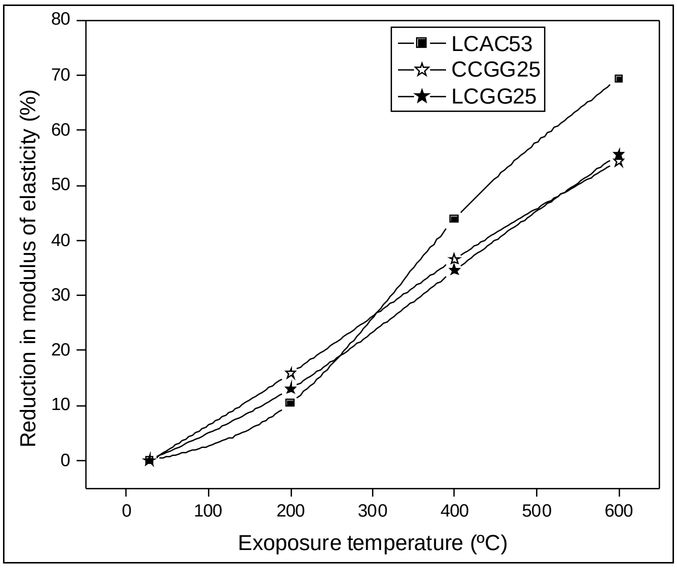

Figure 6.58 Percentage reduction in modulus of elasticity of concrete with temperature after water cooling.

In general, it could be seed from Figures 6.53 to 6.58 that, the percentage reduction in modulus of elasticity of concrete with temperature is less by about 10% up to 200ºC and beyond 200ºC, the rate of reduction is more.

It could be further seen that when OPC without supplementary cementitious materials is used in concrete, concrete with laterite aggregate shows lower rate of reduction in modulus of elasticity when compared to conventional aggregate.

In the present investigation, the percentage reduction in modulus of elasticity of concrete with laterite aggregate and conventional aggregate is in the order of 9 to 11 and 4 to 7 at 200ºC. On the other hand, at 600ºC, the corresponding reductions are in the order of 64 to 70 and 51 to 58 respectively.

It could be further seen from Figures 6.55 to 6.58 that when supplementary cementitious materials are added to concrete, the percentage reduction in the modulus of elasticity of concrete is less compared to concrete with OPC alone. Further, even though marginal (about4%), laterised concrete with supplementary cementitious materials shows lower rate of reduction in modulus of elasticity with temperature compared to corresponding concrete with conventional aggregate.

In the present investigation, at 600ºC, while conventional concrete with supplementary cementitious materials gave a reduction in modulus of elasticity of the order of 51% to 58%, when supplementary cementitious material is used in laterite concrete, the corresponding reduction in modulus of elasticity is in the order of 41% to 56%. It may be noted that when laterite concrete without the addition of cementitious material is used, the percentage reduction in modulus of elasticity at 600ºC is in the order of 64% to 70%. Percentage reduction in the modulus of elasticity of self compacting laterised concrete is compared with the laterised concrete with supplementary cementitious materials in Figures 6.59 and 6.60, respectively for air cooled and water cooled environment.

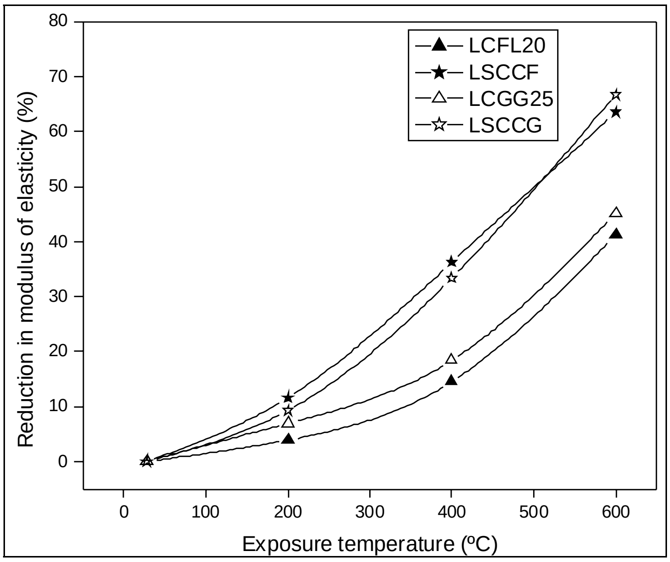

Figure 6.59 Percentage reduction in modulus of elasticity of concrete with temperature after air cooling.

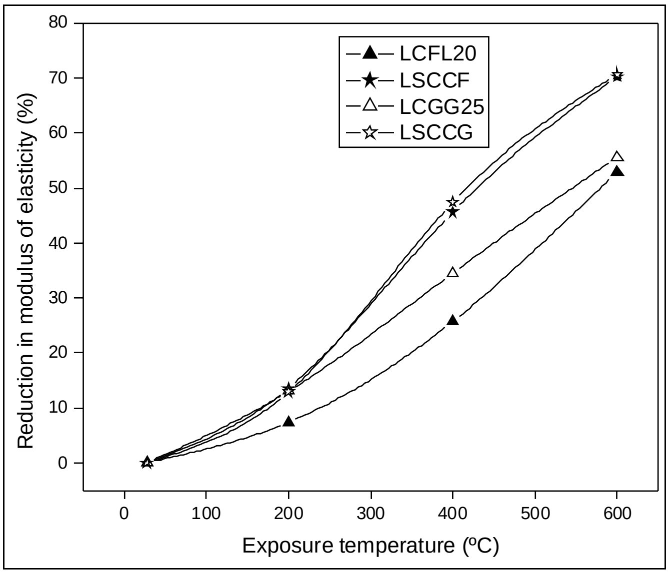

Figure 6.60 Percentage reduction in modulus of elasticity of concrete with temperature after water cooling.

The figures also compare the variation of modulus of elasticity between self compacting concrete and vibrated laterised concrete.

From these figures, it could be seen that self compacting laterised concrete experiences a higher percentage reduction in modulus of elasticity with increase in temperature compared to vibrated laterised concrete. This difference is about 3% to 10% at 200ºC and 20% to 25% at 600ºC for air cooled specimen. This could be due to the higher powder content in the form of additions in concrete and due to the early cracking of self compacting laterised concrete. A similar phenomenon was reported by Hanna Fares et al. [66] and they have suggested that the decrease in modulus of elasticity is due to the cracking of the interfacial zone in concrete.

Figures 6.61 to 6.67 compares the experimental result with the modulus of elasticity predicted based on the equations for conventional concrete proposed by different investigators [63, 60, 62,119, 120, 121, and 122]. Various equations used to compare the experimental results are presented in section 2.7 of Chapter 2.

From Figures 6.61 to 6.67, it can be observed that, even though the available equation for prediction of modulus of elasticity for conventional concrete proposed by various investigators predicts their own test results with a reasonable level of accuracy for temperature higher than 200ºC, all of them underestimate the experimental value in the present study.

It could be noted that the equation proposed by British Standard Institution (BSI) can predict the variation of modulus of elasticity of laterised concrete up to 200ºC with a reasonable level of accuracy. A lower bound prediction equation for variation of modulus

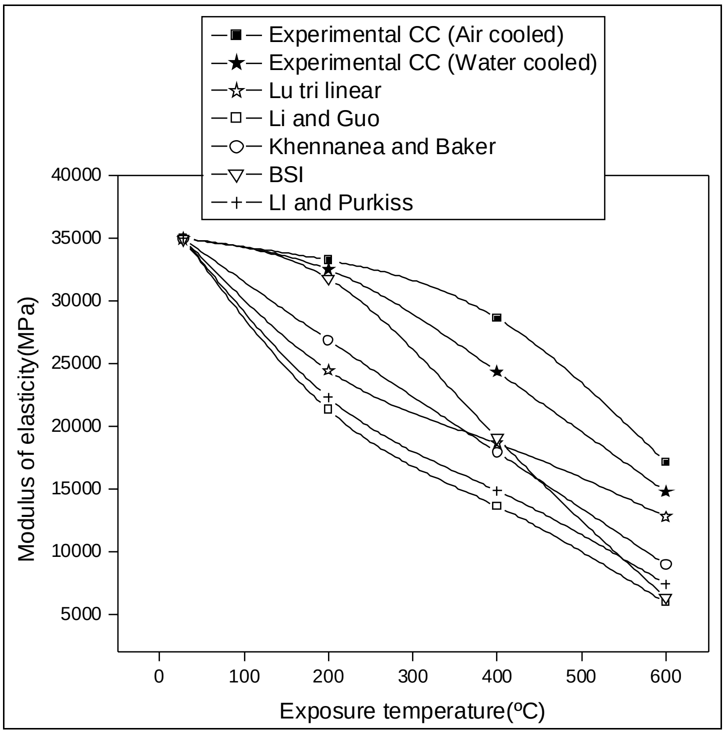

Figure 6.61 Modulus of elasticity-temperature relationship of CC.

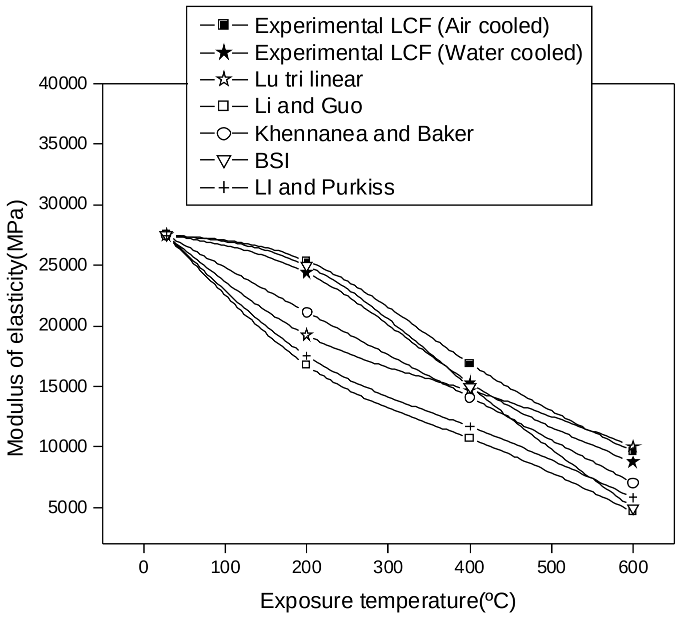

Figure 6.62 Modulus of elasticity-temperature relationship of LCF.

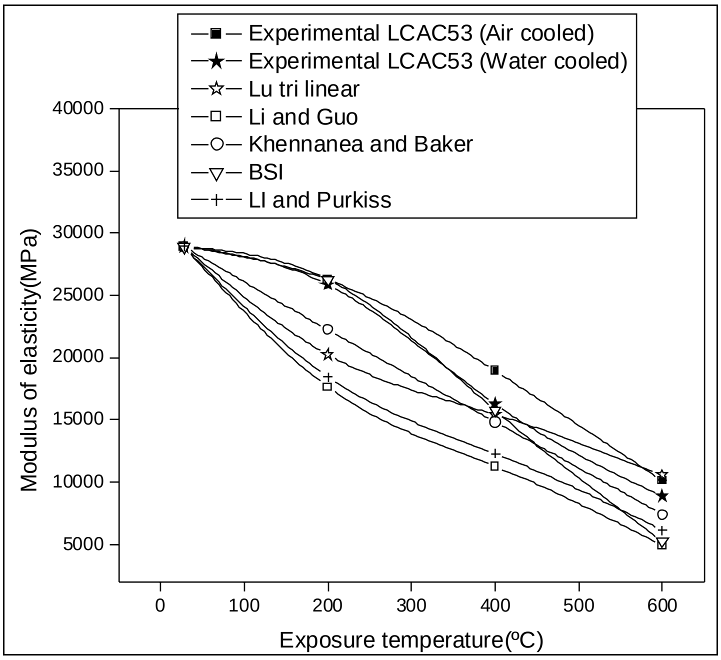

Figure 6.63 Modulus of elasticity-temperature relationship of LCAC53.

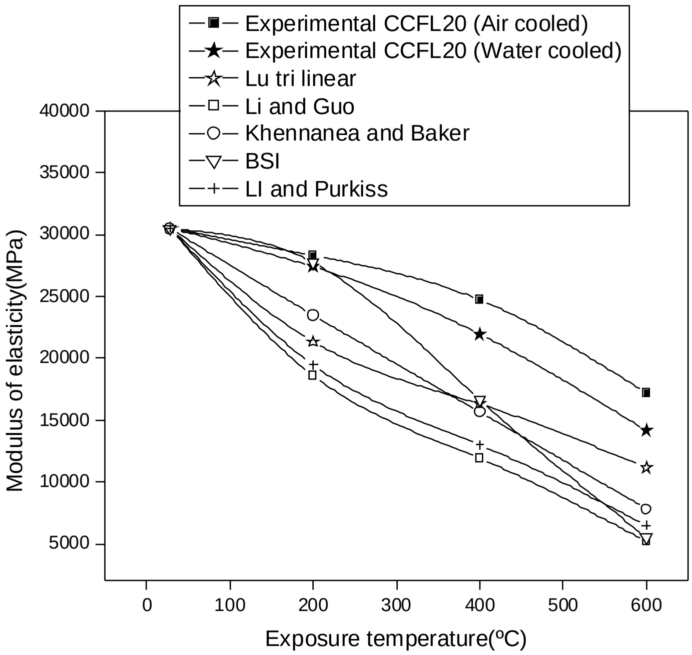

Figure 6.64 Modulus of elasticity-temperature relationship of CCFL20.

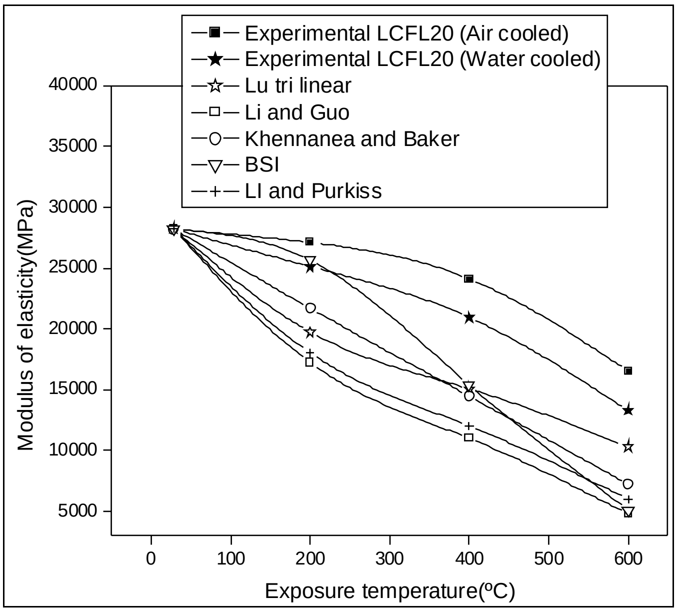

Figure 6.65 Modulus of elasticity-temperature relationship of LCFL20.

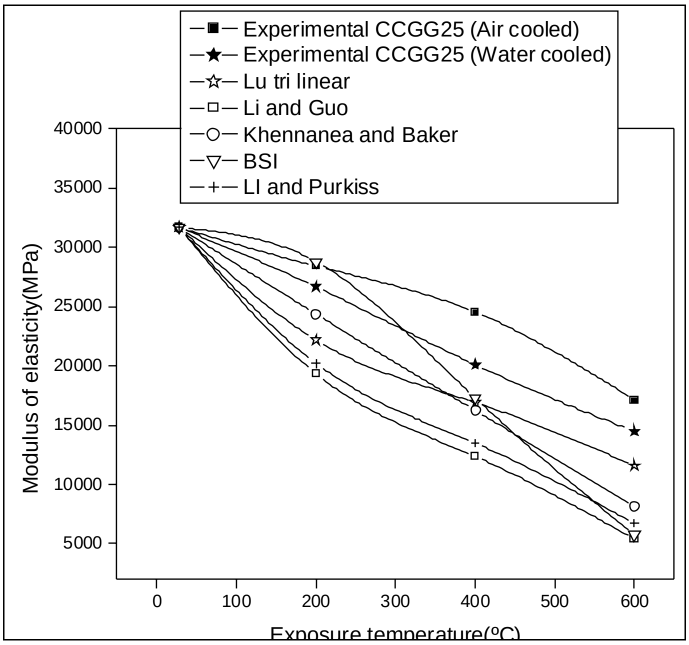

Figure 6.66 Modulus of elasticity-temperature relationship of CCGG25.

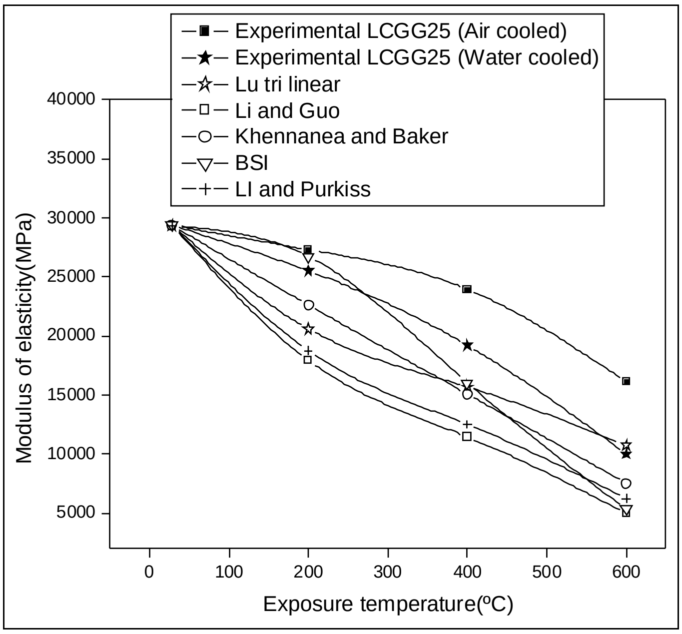

Figure 6.67 Modulus of elasticity-temperature relationship of LCGG25.

of elasticity of laterised concrete having supplementary cementitious materials (fly ash and GGBFS) with temperature has been proposed based on the present study and the same is given as equation 6.3. Figure 6.68 shows the scatter-gram of the test results of modulus of elasticity of laterised concrete having supplementary cementitious materials with temperature.

$$E_{T}= E_{C}\,[1.02- 0.55(T/1000)-0.66(T/1000)^{2}] \quad ....(6.3)$$

6.5 Loss of Unit Mass of Concrete

The unit mass of specimen after heating and subsequent air cooling has been observed and the same is presented in Table 6.6.

Table 6.6 observes that the unit mass of both CC and LC decreases with increase in temperature, which is expected when concrete is exposed to high temperature and is attributed to the loss of moisture from concrete.

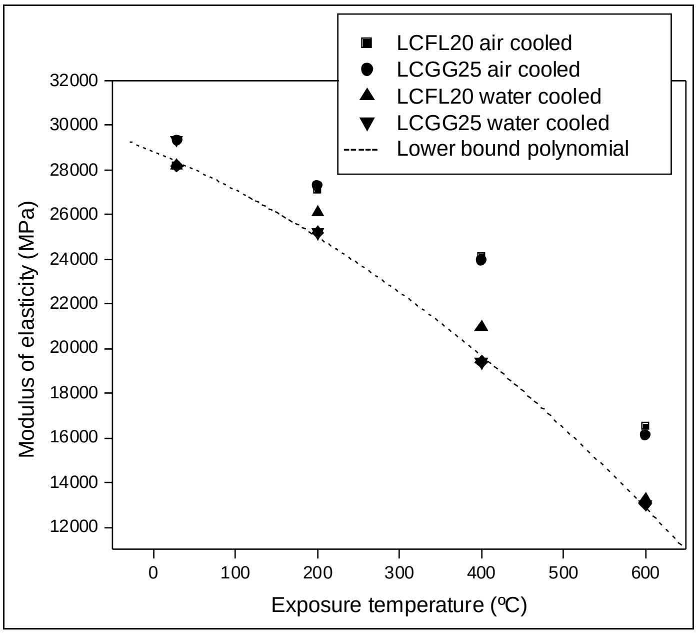

Figure 6.68 Scatter-gram of the test results of modulus of elasticity of laterised concrete having supplementary cementitious materials with temperature.

Figure 6.69 depict the percentage reduction in unit mass of concrete exposed to different temperature with respect to the unit mass of concrete at ambient temperature.

From Figure 6.69 the trend of percentage reduction of unit weight for all types of concrete is similar. It can be observed that up to 200ºC the loss of mass increases gradually and is primarily due to the loss of free moisture. Between 200 and 400ºC the rate of loss of mass is less and is attributed to the loss of physically adsorbed water. From 400ºC onwards rate of loss of mass further increased and is attributed due to the loss of chemically combined water from the hydrated cement products. This observation is in agreement with the findings of Rao et al.[20]. Siddique and Kauduer [81] also reported that at temperatures between 200 and 350ºC, the loss of mass is not very significant and the same is true here also.

Table 6.6 Unit mass of concrete after the exposure to elevated temperature

Sl. No.	Concrete type (M25 grade)	Unit mass (kg/m^3)							
		Ambient 28 °C		Exposure temperature(°C)					
				200		400		600	
		Mean	SD	Mean	SD	Mean	SD	Mean	SD
1	CC	2470	5.57	2432	17.52	2419	16.37	2362	21.28
2	LCF	2458	7.21	2421	17.06	2410	17.44	2357	23.64
3	LCAC53	2440	7.00	2415	21.79	2409	15.72	2360	16.64
4	CCFL20	2486	7.94	2445	12.12	2424	19.08	2353	16.64
5	LCFL20	2440	9.54	2418	9.17	2412	15.87	2350	16.09
6	CCGG25	2480	12.29	2442	15.13	2432	22.61	2347	11.00
7	LCGG25	2460	13.23	2424	30.35	2414	12.17	2329	31.00
8	LSCCF	2498	18.36	2468	12.49	2460	22.54	2399	15.87
9	LSCCG	2509	17.35	2474	12.77	2445	12.77	2394	26.06

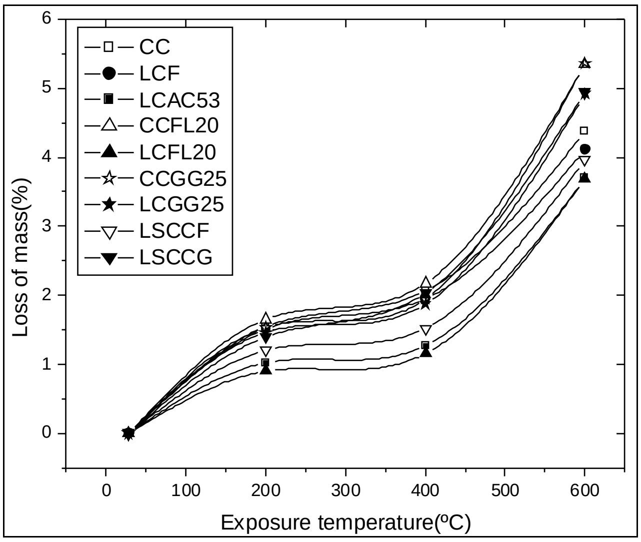

Figure 6.69 Percentage variation of unit mass of concrete exposed to high temperature.

It can be observed that the loss of unit mass with temperature is low for concrete with laterised all-in aggregate (LCAC53) as well as for laterised concrete with partial replacement of cement with fly ash (LCFL20). This may be one of the reasons for the no crack formation of these (LCAC53 and LCFL20) concrete even when exposed to a temperature of 800°C. It is clear that loss of chemically combined water in concrete is one of the major factor controls the cracking when exposed to high temperature.

6.6 Shear Strength of Laterised Self Compacting Concrete (LSCCF)

Figure 6.70 shows the variation of average shear strength of LSCCF with age of concrete. The average shear strength on 28th day is 5.20 MPa. The real strength of most materials like concrete may be determined by their tensile strength, which causes a first fracture of the element under various stress conditions. Yoshitake [123] already observed that, the shear cracking strength of a concrete element is almost equal to 80% of splitting tensile

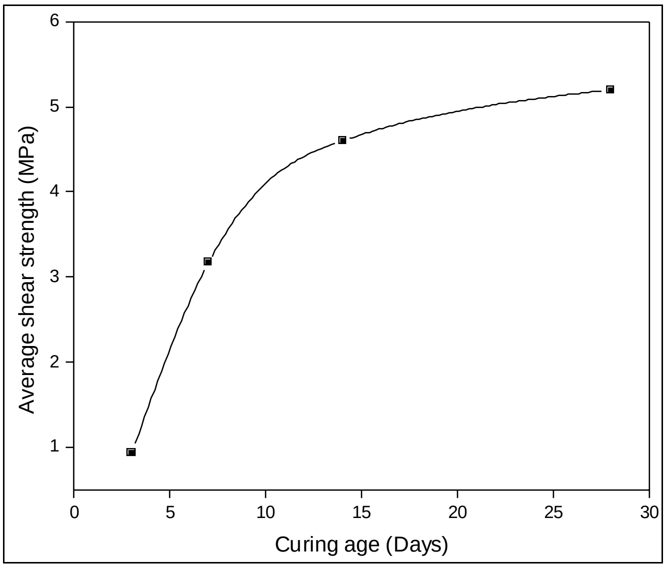

Figure 6.70 Average shear strength-curing age relationship

strength on an average, hence the tensile crack start before shear crack originate. So shear crack in a concrete element may not always lead to failure but such cracks influence the service life of concrete structures.

Based on the present investigation, the approximate relationship between average shear strength and compressive strength of LSCCF can be written as $\tau_c = 0.9\sqrt{f_{ck}}$. However, further study is required to have a more reliable relationship for average shear strength.

6.7 Cracking Behaviour of Concrete

The surface cracking of CC and LC specimen has been observed by visual inspection. The entire specimens were checked before and after heating. Cracks were observed immediately after removal of specimen from the furnace. It was observed that both CC and LC specimen did not show any surface cracking up to 400°C. However, at 600°C, all specimens (CC series) developed one or more major cracks and few minor cracks. When specimen was cooled by sprinkling water, several additional distributed cracks were developed due to the thermal shock induced by water spray. On the other hand, at 600ºC concrete with laterite fine aggregate (LCF) as well as laterite all in aggregate (LCAC53) did not show any cracking, even in water cooled environment. This behavior of LC may be due to the thermal stability of laterite aggregate. Figures 6.71 and 6.72 show the typical crack pattern on CC specimen after heating to 600°C.

Figure 6.71 Typical major crack in CC heated to 600°C-overall view.

It may be further noted that, when supplementary cementitious materials are added in cement concrete, specimen did not crack even up to 600ºC. Similar enhanced crack resistance behavior of concrete with supplementary cementitious material has been reported elsewhere also [59]. When the specimen with supplementary cementitious materials were heated further to 800ºC, the control concrete (CCFL and CCGG) developed distributed cracks. On the other hand, laterite concrete with supplementary

Figure 6.72 Typical major crack in CC heated to 600°C-closer view.

cementitious materials (LCFL and LCGG) did not show any crack even when exposed to water spray. Figure 6.73 shows the typical crack pattern of CCFL specimen heated to 800ºC and compares the uncracked surface of corresponding specimen when exposed to a temperature of 600°C.

Figure 6.73 Comparison of crack pattern of CCFL specimen heated to 800ºC and 600ºC

Laterised self compacting concrete (LSCCF and LSCCG) did not show any crack up to 400ºC. However at 600ºC, they developed distributed cracks. The early development cracks in LSCC are due to the presence of higher percentage of additions (fly ash and GGBFS). Figures 6.74 and 6.75 shows crack pattern of LSCCF and LSCCG at 600ºC. It may be noted that when water is sprayed, additional cracks were formed due to thermal shock. Figure 6.76 shows typical crack pattern of LSCC at 600ºC, after spraying water. Based on the present study, it could be observed that, whole CC developed one or two large major wider cracks, laterised self compacting concrete develops only distributed hair line cracks.

Figure 6.74 Typical crack pattern on LSCCF at 600°C under air cooling

In conclusion, it may be stated that laterite concrete with additional supplementary cementitious materials (fly ash and GGBFS) enhances the crack resistance property of concrete up to 800ºC. Even though laterite self compacting concrete develops cracks at

Figure 6.75 Typical crack pattern on LSCCG at 600°C under air cooling-closer view

600ºC, they are distributed hair line cracks only and behave better compared to normal concrete, which develops one or two major cracks at 600°C.

Figure 6.76 Typical crack pattern on LSCCF at 600°C under water cooling

6.8 Colour Change of Laterised Concrete at Elevated Temperature

Figure 6.77 shows the colour change in laterised concrete, when exposed to different temperature levels. Laterised concrete, at ambient temperature, was grayish in colour over its cement matrix region and the aggregates were brownish in colour. There was no appreciable colour change in the cement matrix up to 200^{o}C. However, at 400^{o}C, the colour of cement matrix tends to change towards pale grey and at 600^{o}C, the entire cement matrix turned to pale grey. No remarkable colour change has been observed in laterite aggregate up to 600^{o}C. In the case of control specimens minor colour change only could be observed up to 600^{o}C.

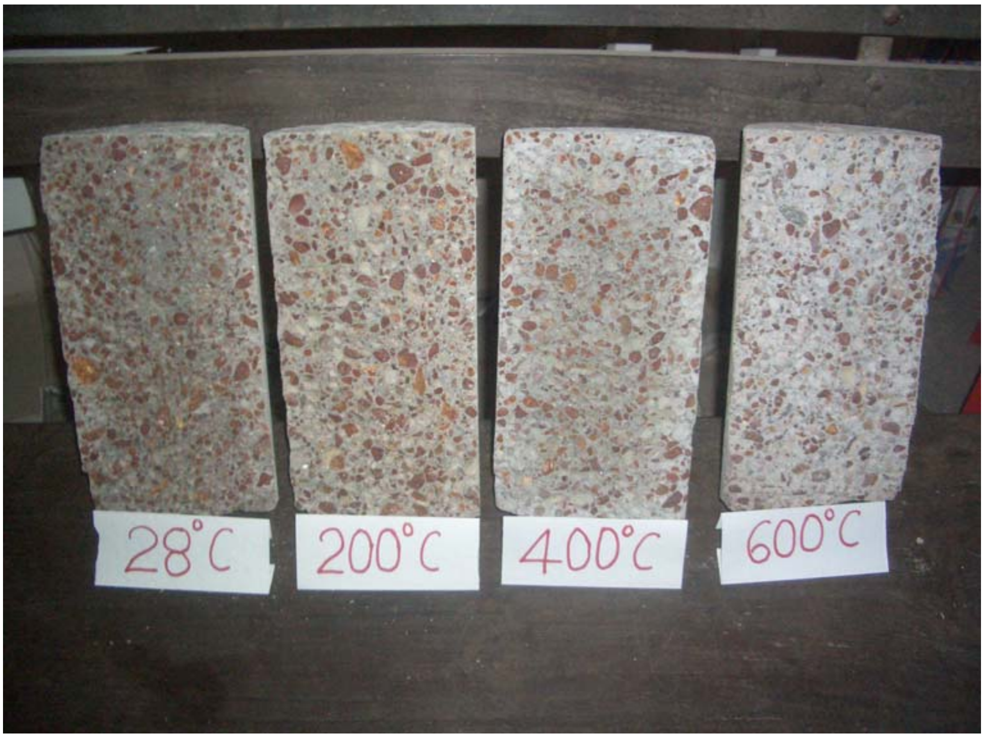

Figure 6.77 Typical colour change of laterised concrete at elevated temperature

6.9 Behaviour of Ferrocement Element with Laterised Self Compacting Concrete Exposed to Elevated Temperatures

It has been observed that laterised concrete behaves better against cracking at elevated temperature compared to normal concrete. Hence it has been decided to study the behaviour of ferrocement element with LSCC, so that its usefullness as a protection material against fire exposure can be investigated.

Ferrocement elements are generally considered for jacketing of structural members to protect from fire exposure. Since development of crack in such elements, which are thin in nature, may lead to lose of integrity. Hence, in the present study, the flexural strength of ferrocement elements at the development of first crack (Modulus of rupture) has been considered as the limiting criterion for the integrity of these elements when exposed to high temperature. Greepala and Nimityongskul[82] reported that volume fraction of mesh reinforcement in ferrocement elements has not much significance on the mechanical properties when exposed to high temperature. So single layer wire mesh (4mm square mesh openings, 0.78mm wire diameter and weight 0.265kg/m^2) has been considered for the present study. The variable considered is the cover to mesh reinforcement. Specimen were prepared by keeping the area of reinforcement constant (340mm×100mm), the covers to mesh on all sides were varied from 10mm to 50mm at an intervel of 10mm and they were correspondingly designated as SFE1, SFE2, SFE3, SFE4 and SFE5. The volume fraction of SFE1 series is 0.33%.

The ferrocement elements were heated to temperature of 200°C, 400°C and 600°C and cooled to ambient temperature before testing. Typical deformation behaviour of ferrocement element(SFE1) with LSCCF is shown in Figure 6.78.

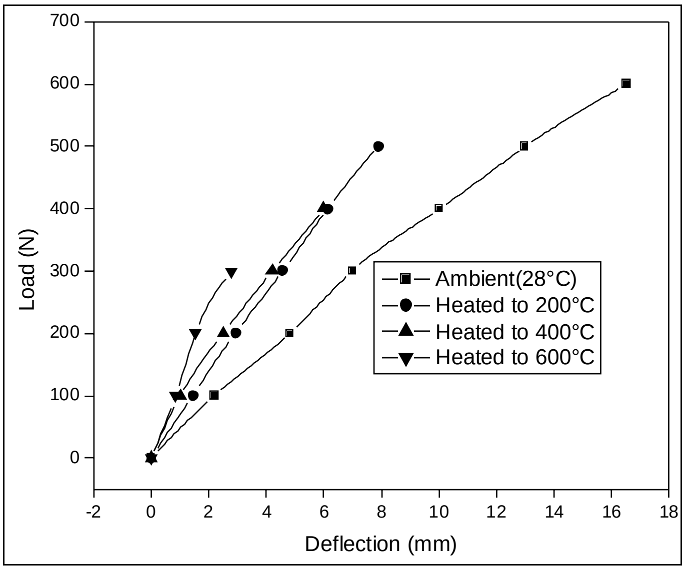

Figure 6.78 Load - deformation diagram of ferrocement element (SFE1) exposed to elevated temperature.

From this figure, it is clear that as temperature increases, the specimens become stiffer.

Table 6.7 presents the modulus of rupture value of ferrocement elements made with LSCCF. From Table 6.7, it could be seen that, as temperature increases from ambient (28°C) to 600°C, there is a reduction of modulus of rupture value of ferrocement LSSCF. However the rate of reduction of modulus of rupture with temperature is more or less same for specimens having different cover to the mesh reinforcement. In the present study, for specimens with cover varying from 10mm to 50mm, the percentage reduction in modulus of rupture at 600°C was between 45% and 50%.

Further, the modulus of rupture of specimen decreases as the cover of reinforcement increases. It could be noted that the rate of reduction in modulus of rupture of specimen with variation in cover to reinforcement is more or less the same for all exposure temperature. In the present study, as cover is increased from 10mm to 50mm, the modulus of rupture reduced

Table 6.7 Modulus of rupture of ferrocement element at elevated temperature

Sl. No.	Specimen type	Modulus of rupture (MPa)															
		Air cooled								Water cooled							
		Exposure temperature(°C)								Exposure temperature(°C)							
		28		200		400		600		28		200		400		600	
		Mean	SD	Mean	SD	Mean	SD	Mean	SD	Mean	SD	Mean	SD	Mean	SD	Mean	SD
1	SF1	17.33	1.83	14.44	1.18	11.55	1.36	8.66	0.67	17.33	1.83	11.33	1,60	8.17	0.41	5.89	0.85
2	SF2	10.55	0.13	9.86	0.81	7.27	0.51	5.38	0.81	10.55	0.13	8.96	1.17	4.98	0.39	3.58	0.27
3	SF3	7.90	0.82	7.40	0.58	6.70	0.21	4.05	0.18	7.90	0.82	6.64	0.31	4.50	0.47	3.20	0.42
4	SF4	5.30	0.61	4.00	0.19	3.30	0.17	2.73	0.16	5.30	0.61	3.60	0.15	2.40	0.13	2.16	0.30
5	SF5	4.10	0.37	3.70	0.33	2.90	0.40	2.20	0.15	4.10	0.37	2.80	0.42	1.92	0.18	1.80	0.13

by 76% in the case of specimen at ambient temperature and 75% for specimen exposed to 600°C. So it may be concluded that ferrocement element LSCCF having less cover to mesh reinforcement (10mm in the present study) yields higher modulus of rupture value at all temperature levels.

Figure 6.79 shows the scatter-gram of the test results of modulus of rupture of ferrocement element made with LSCCF having fly ash as addition with temperature. A lower bound prediction equation for variation of modulus of rupture of ferrocement element made with laterised self compacting concrete having fly ash as addition with temperature has been proposed based on the present study and the same is given as equation 6.4 for polynomial fit.

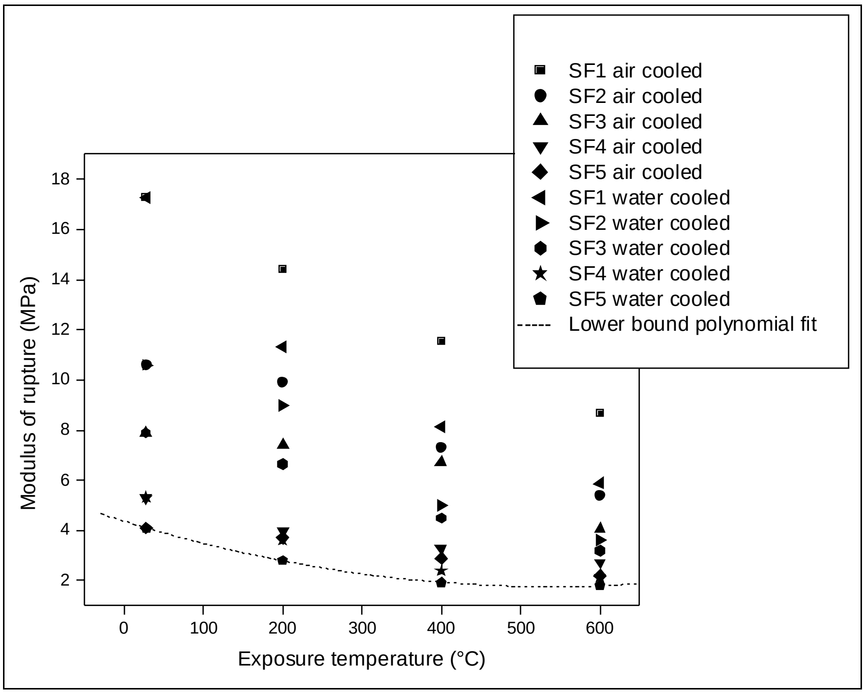

Figure 6.79 Scatter-gram of the test results of modulus of rupture of ferrocement element made with LSCCF having fly ash as addition with temperature.

$f_{bT} = f_b\,[1.07-2.37(T/1000)-2.21(T/1000)^2]$ for 28< T >600............(6.4)

6.10 Cracking of Ferrocement Element

The surface cracking of ferrocement element specimen has been observed by visual inspection. The entire specimen was checked before and after heating. Cracks were observed immediately after removal of specimen from the furnace. It was observed that ferrocement specimen did not show any surface cracking up to 400°C. However, at 600°C, all specimens developed distributed hair line cracks. It has been observed in the present study that the cover to mesh reinforcement has no significant influence on crack initiation and/or its distribution. When specimen was cooled by sprinkling with water, several additional distributed cracks were developed due to the thermal shock induced by water spray. Figure 6.80 shows typical crack pattern on ferrocement specimen heated to 600°C and cooled with sprinkling of water.

Figure 6.80 Typical crack pattern on specimen heated to 600°C and cooled with sprinkling of water.

6.11 Concluding Remarks

From the present study following conclusions can be drawn;

- Mineral admixture (supplementary cementitious materials) significantly improves the residual compressive strength of concrete after exposure to high temperature.
- Addition of supplementary cementitious materials (fly ash/GGBFS) in concrete improves the resistance against cracking when exposed to high temperature.
- Compared to laterised concrete with ordinary portland cement alone, when fly ash is added as addition, the residual compressive strength after exposure to high temperature improves. This is true for modulus of elasticity also.
- Compared to fly ash, if GGBFS is added in laterised concrete, the residual compressive strength is low after exposure to high temperatures.
- The available equations for the prediction of residual compressive strength and split tensile strength of normal concrete after exposure to high temperature yields poor strength prediction in the case of laterised concrete.
- In general, the split tensile strength of conventional concrete reduces with exposure temperature. When sand is replaced fully with laterised fine aggregate, the rate of reduction in split tensile strength of concrete is low. When both sand and coarse aggregate in conventional concrete is replaced with laterised all-in aggregate, the rate of reduction becomes still lower.

- The laterised concrete with supplementary cementitious materials shows lower rate of reduction in modulus of elasticity with temperature compared to corresponding cement with conventional aggregate, even though marginal (about4%).
- The reduction in compressive strength with increase in temperature is less in laterised self compacting concrete when compared with vibrated laterised concrete.
- The reduction in tensile strength and modulus of elasticity with increase in temperature is more in laterised self compacting concrete when compared with vibrated laterised concrete. This is primarily due to the presence of large quantity of additions in the former concrete when compared with the later.
- A lower bound equation to predict the modulus of elasticity of laterised concrete at elevated temperature has been proposed based on the present study.
- When specimen was cooled by sprinkling water, several additional distributed cracks were developed due to the thermal shock induced by water spray. On the other hand, at 600ºC concrete with laterite fine aggregate (LCF) as well as laterite all-in aggregate (LCAC53) did not show any cracking, even in water cooled environment.
- Laterised concrete with additional supplementary cementitious materials (fly ash and GGBFS) does not crack when exposed to a temperature of up to 800°C.
- Laterised self compacting concrete develops distributed hair line cracks at 600°C.

- Large quantity of supplementary cementitious material in concrete may lead to early thermal cracks.
- The loss of unit mass increases gradually up to 200ºC and is primarily due to the loss of free moisture.
- The loss of unit mass when exposed to between 200°C to 400°C is low and attributed primarily to loss of physically adsorbed water. From 400ºC onwards considerable reduction were observed and is attributed to the loss due to the chemically combined water of the hydrated cement products.
- The loss of unit mass with temperature is low for concrete with laterised all in aggregate (LCAC53) as well as for laterised concrete with partial replacement of cement with fly ash (LCFL20). This may be one of the reasons for the no crack formation of these (LCAC53 and LCFL20) concrete even when exposed to a temperature of 800°C. It is clear that loss of chemically combined water in concrete is one of the major factor controls the cracking when exposed to high temperature.
- No remarkable colour change has been observed in laterite aggregate up to 600^{o}C. In the case of control specimens minor colour change only could be observed up to 600^{o}C.
- Ferrocement specimen did not show any surface cracking up to 400^{o}C. However, at 600^{o}C, all specimens developed distributed hair line cracks. When specimen was cooled by sprinkling with water, several additional distributed cracks were developed due to the thermal shock induced by water spray.

Chapter -7

CONCLUSIONS AND SCOPE FOR FUTURE STUDY

7.1 General

A systematic study carried out to understand the possibility of using weathered laterite aggregate for preparing concrete is presented in this thesis. Studies have been made to understand the performance of concrete when weathered laterite fine and all-in aggregates are used in both conventional and self compacting normal strength concrete. The study has been extended to reveal the thermal behaviour of both the types of laterised concretes and to examine its suitability as a fire protection material.

A basic study on laterised concrete with variables like source of laterite aggregate, grades of OPC and types of supplementary cementitious materials has been done to arrive at an amicable combination of various constituents in laterised concrete.

A mix design procedure has been proposed for making normal strength laterised self compacting concrete based on trial mixes and the same has been validated.

The properties of laterised concrete have been examined by considering different parameters like exposure temperature and cooling environments.

The performance of ferrocement flexural elements with laterised self compacting concrete has also been studied by varying the cover to mesh reinforcement, exposure temperature and cooling environments.

7.2 Conclusions

Based on the present study, following major conclusions have been derived.

- The physical properties of weathered laterite aggregates collected from different parts of the state of Kerala are more or less the same.
- In general, concrete with laterised all-in aggregate shows lower slump value, even though marginal, compared to the corresponding conventional concrete.
- The slump value of laterised concrete can be increased by partial replacement of cement with fly ash as well as GGBFS.
- In the present study, compared to the slump value of laterised concrete, when 20% of OPC was replaced by fly ash, the slump value was seen enhanced by 19%.
- In general, the compressive strength of concrete with weathered laterite all-in aggregate is lower compared to a corresponding conventional concrete. In the present study, a 9% lower strength has been observed for laterised concrete when compared to the strength of M25 grade conventional concrete.
- When cement was partially replaced with fly ash or GGBFS, there was no significant reduction in cube compressive strength for both laterised and conventional concrete up to a certain replacement level. Beyond this level, a drastic reduction in cube compressive strength has been observed for both the types of concrete. In the present study, it was noticed that ideally, 20% of OPC can be replaced by fly ash or 25% of OPC by GGBFS in conventional as well as laterised concrete without the compressive strength being affected. Such replacement results in the production of more economical concrete.

- A mix design method has been proposed for nominal strength (M20 to M40) laterised self compacting concrete (LSCC) with fly ash or GGBFS as addition.
- The proposed mix design method for LSCC has been validated by comparing the experimental results with appropriate standards.
- Unlike conventional SCC, laterised self compacting concrete demands large quantity of additions (fly ash or GGBFS) to achieve required flow properties.
- Even though the strength of laterised concrete is lower at ambient temperature compared with corresponding conventional concrete, at exposure temperature, above 400°C, there is a high rate of reduction in strength for conventional concrete, primarily due to the early development of micro cracks in conventional concrete.
- Addition of mineral admixture (fly ash or GGBFS) reduces the loss of strength of both laterised and conventional concrete at high temperature.
- Unlike the strength of laterised concrete with GGBFS at ambient temperature, the residual compressive strength of laterised concrete as well as laterised self compacting concrete with fly ash is higher at higher temperatures.
- The available equations for the prediction of compressive strength, split tensile strength and modulus of elasticity of normal concrete after exposure to high temperatures yields poor prediction in the case of laterised concrete.
- Lower bound equations to predict the cube compressive strength, split tensile strength and modulus of elasticity of laterised concrete at high temperatures has been proposed based on the present study.

- The loss of unit mass of laterised concrete when exposed to a temperature level between 200 ºC and 400ºC is not significant and is attributed to the physically adsorbed water. However, for exposure temperature above 400ºC, considerable reduction in unit mass has been observed and is attributed to the loss of chemically combined water present in hydrated cement products.
- While conventional concrete develops distributed cracks at 600°C, laterised concrete did not show any cracks at 600°C, even in water cooled environment.
- When mineral admixture (fly ash or GGBFS) was added, the conventional concrete did not crack up to 600°C and laterised concrete did not develop any crack even at 800°C. However laterised self compacting concrete developed distributed hair line cracks at 600°C.
- Ferrocement flexural specimen did not show any surface cracking up to 400°C. However, at 600°C, all specimens developed distributed cracks. When specimens were cooled by sprinkling with water, several additional distributed cracks were developed due to the thermal shock induced by water spray.
- The loss of chemically combined water in concrete is one of the major factors that control the cracking of concrete specimen when exposed to high temperature.
- A lower bound prediction equation for the modulus of rupture of ferrocement flexural elements (made with laterised self compacting concrete having fly ash as addition) with temperature has been proposed based on the present study.

In conclusion, the combined use of weathered laterite aggregate and additions as fly ash or GGBFS in laterised concrete (LC) and laterised self compacting concrete (LSCC) can produce green and economical concrete which has better physical properties compared to conventional concrete when exposed to high temperatures. Hence LC and LSCC are suitable as fire protection materials compared to conventional concrete.

7.3 Scope for Further Studies

Based on the present study and conclusions drawn, following areas have been identified for the further studies.

- The present lower bound equations have been proposed based on M25 grade laterised concrete only. These equations need to be refined by considering different grades of laterised concrete.
- Detailed study on reinforced laterised concrete and laterised self compacting concrete may be carried out for confirming the general purpose application of LC and LSCC.
- Use of laterite aggregate in Geo polymer concrete can be investigated.
- Study of alkali-silica reaction on LC and LSCC exposed to high temperature can be carried out.

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LIST OF PUBLICATIONS

Publications based on the research works

Papers in Journals

- Mix Design Methodology for Laterized Self Compacting Concrete and its Behaviour at Elevated Temperature, *Journal of Construction and Building Materials,* Elsevier, (Under review).
- Behaviour of Laterised Concrete Exposed to Elevated Temperatures, *Journal of Building and Environment.* Pragaman (Communicated).
- Behaviour of Laterised Concrete with Mineral Admixtures (Fly ash and GGBFS) Exposed to Elevated Temperatures. *The Indian Concrete Journal*, (Communicated)

Papers in conference

- Influence of GGBFS in laterised concrete and its effects at elevated temperatures. *Proceedings of the 4th National conference on Recent Advances in Civil Engineering (RACE), Division of Civil Engineering, CUSAT, Kochi, September 16-18, 2010,* Pp 83-88.

Mike Forde m.forde@ed.ac.uk
to george_m, me

Ms. Ref. No.: CONBUILDMAT-D-11-00734
Title: MIX DESIGN METHODOLOGY FOR LATERIZED SELF COMPACTING
CONCRETE AND ITS BEHAVIOUR AT ELEVATED TEMPERATURE
Construction & Building Materials

Dear George,

Reviewers have now commented on your paper. You will see that they are advising that you revise your manuscript. If you are prepared to undertake the work required, I would be pleased to consider your paper for publication.

For your guidance, reviewers' comments are appended below.

To submit a revision, please go to http://ees.elsevier.com/conbuildmat/ and login as an Author.

Yours sincerely,

Professor Mike Forde, PhD

Editor-in-Chief
Construction & Building Materials

Reviewers' comments:

The technical content of the paper is good. However, the following suggested minor corrections should be effected before publication:

Page 2:

Remove the brackets in lines 31 and 33, then the sentence will read "In places where compaction and placing are difficult, such as jacketing of structural elements for fire protection, backfilling near retaining structures et cetera......."

Page 6:

Line 45, instead of "......test method has been used here," should read ".....test method was used in this work."

Page 7:

Line 38, instead of ".....obstuction are assessed....," should read "....obstruction were assessed...."

Line 40, instead of "......filling ability have been measured...," should read ".....filling ability were measured...."

APPENDICES

Appendix -A

TEST PROCEDURES AND SPECIFICATIONS FOR SCC

A1-Test procedure of Slump flow + T_{500} test

- Place the cleaned base plate in a stable and level position.
- Fill the bucket with 6 to7 liters of representative fresh SCC and let the sample stand still for about 1 minute (± 10 seconds).
- During the 1 minute waiting period, pre-wet the inner surface of the cone and the test surface of the base plate using the moist sponge or towel, and place the cone in the centre on the 200 mm circle of the base plate and put the weight ring on the top of the cone to keep it in place.
- Fill the cone with the sample from the bucket without any external compacting action such as rodding or vibrating. The surplus concrete above the top of the cone should be struck off, and any concrete remaining on the base plate should be removed.
- Check and make sure that the test surface is neither too wet nor too dry. No dry area on the base plate is allowed and any surplus of the water should be removed – the moisture state of the plate has to be ‘just wet’.
- After a short rest of the test surface lift the cone perpendicular to the base plate in a single movement, in such a manner that the concrete is allowed to flow out freely without obstruction from the cone, and start the stopwatch the moment the cone looses contact with the base plate.

- Stop the watch when the front of the concrete first touches the circle of diameter 500 mm. The corresponding time is recorded as the T_{500} value. The test is completed when the concrete flow has ceased.
- Measure the largest diameter of the flow spread D_1 and the one perpendicular to it D_2 in millimeters.
- The slump flow spread S expressed in millimeters is the average of the diameters D_1 and D_2.

Figure 3.7 depicts the method of arriving at the slump flow value.

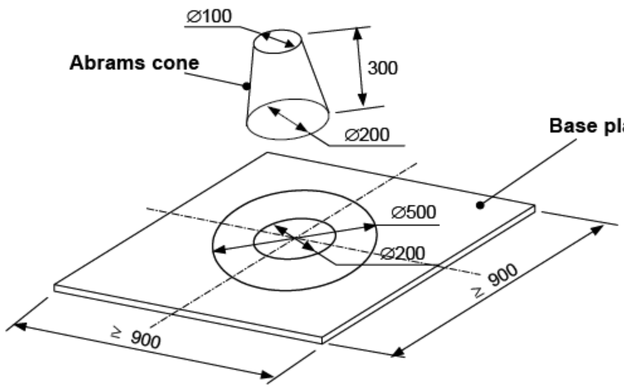

Figure A1 Method of arriving T_{500}

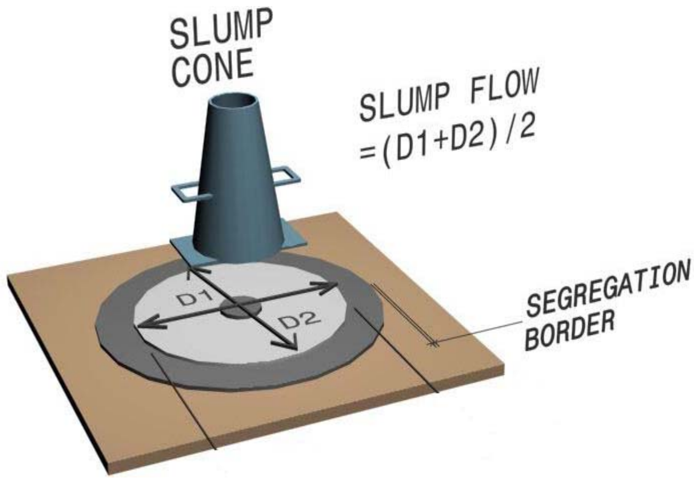

Figure A2 Method of arriving slump flow

A1.1 Slump flow classification

Slump-flow value describes the flowability of a fresh mix in unconfined conditions. It is a sensitive test that will normally be specified for all SCC, as the primary check that the fresh concrete consistence meets the specification. Visual observations during the test and/or measurement of the T_{500} time can give additional information on the segregation resistance and uniformity of each delivery.

The following are typical slump-flow class for a range of applications;

SF1 (550-650mm) is appropriate for:

- Unreinforced or slightly reinforced concrete structures that are cast from the top with free displacement from the delivery point (e.g. housing slabs)
- Casting by a pump injection system (e.g. tunnel linings)
- Sections that are small enough to prevent long horizontal flow (e.g. piles and some deep foundations)

SF2 (660-750mm) is suitable for many normal applications (e.g. walls, columns) SF3 (760-850mm) is typically produced with a small maximum size of aggregates (less than 16mm) and is used for vertical applications in very congested structures, structures with complex shapes, or for filling under formwork. SF3 will often give better surface than SF2 for normal vertical applications but segregation resistance is more difficult to control.

Target values higher than 850mm may be specified in some special cases but great care should be taken regarding segregation and the maximum size of aggregate should normally be lower than 12mm

A2-Test procedure of L-box test

- Place the L-box in a stable and level position.

- Fill the vertical part of the L-box, with the extra adapter mounted, with 12.7 liters of representative fresh SCC.
- Let the concrete rest in the vertical part for one minute. During this time the concrete will display whether it is stable or not (segregation).
- Lift the sliding gate and let the concrete flow out of the vertical part into the horizontal part of the L-box.
- When concrete has stopped moving, measure the depth from the top of L-box to the top of concrete surface at three points, namely on either ends and at the middle in millimeters.
- Calculate the mean of the three measurements, which is referred as Δh

The passing ratio P_L (or blocking ratio B_L) is calculated using equations A.1.

$$P_L = \frac{H}{H_{max}} \quad \text{.......} \quad (A.1)$$

where, H_{max} = 91 mm and $H = 150 - \Delta h$

Figure 3.8 depicts the typical L-box for measuring passing ability.

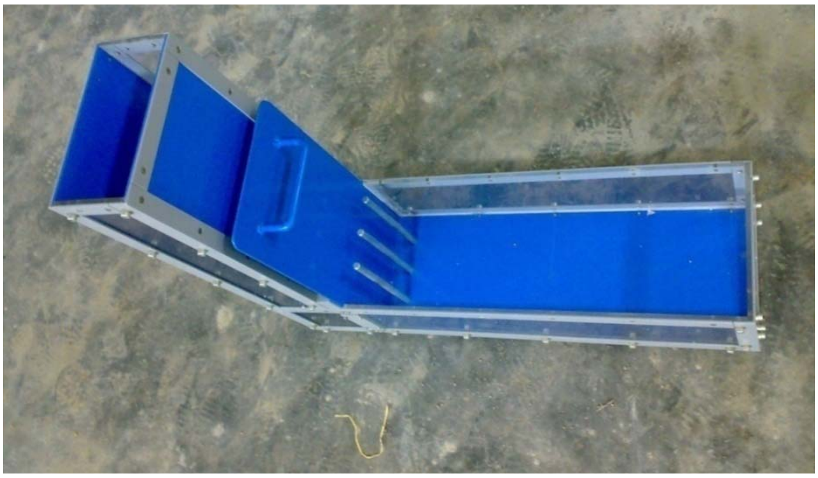

Figure A3 Typical L-box

A2.1 Passing ability classifications

Passing ability describes the capacity of the fresh mix to flow through confined spaces and narrow openings such as areas of congested reinforcement without segregation, loss of uniformity or causing blocking. In defining the passing ability, it is necessary to consider the geometry and density of the reinforcement, the flowability/ filling ability and the maximum aggregate size.

The defining dimension is the smallest gap (confinement gap) through which SCC has to continuously flow to fill the formwork. This gap is usually but not always related to the reinforcement spacing. Unless the reinforcement is very congested , the space between the reinforcement and formwork cover is not normally taken into account as SCC can surround the bars and does not need to continuously flow through these spaces.

Table A.1Passing ability class and values

Sl. No.	Class	Passing ratio
1	PA1	≥0.80 with 2 rebars
2	PA2	≥0.80 with 3 rebars

Examples of passing ability specifications are given below;

- PA1- Structures with a gap of 80mm to 100mm, (e.g. housing, vertical structures).
- PA2- Structures with a gap of 60mm to 80mm, (e.g. civil engineering structures).

For thin slabs where the gap is greater than 80mm and other structures where the gap is greater than 100mm no specified passing ability is required.

For complex structures with gap less than 60mm, specific mock-up trials may be necessary.

A3-Test procedure of V-funnel test

- Place the cleaned V-funnel vertically on a stable and flat ground, with the top opening horizontally positioned.
- Wet the interior of the funnel with a moist sponge or towel and remove the surplus water. The inner side of the funnel should be ‘just wet’.
- Close the gate and place a bucket under it in order to retain the concrete to be passed.
- Fill the funnel completely with a representative sample of SCC without applying any compaction or rodding.
- Remove any surplus of concrete from the top of the funnel using the straightedge.
- Open the gate after a waiting period of 10 seconds. Start the stopwatch at the same moment the gate is opened.
- Look inside the funnel and stop the time at the moment when clear space is visible through the opening of the funnel. The stopwatch reading is recorded as the V-funnel flow time, t_V in seconds.

Figure A.4 depicts the typical V-funnel for measuring filling ability.

Figure A.4 Typical V-funnel

A3.1 Viscosity classification

Viscosity can be assessed by the T_{500} time during the slump-flow test or assessed by the V-funnel flow test time. The time value obtained does not measure the viscosity of SCC but is related to it by describing the rate of flow. Concrete with a low viscosity will have a very quick initial flow and then stop. Concrete with a high viscosity may continue to creep forward over an extended time.

Viscosity (low or high) should be specified only in special cases such as those given below. It can be useful during mix development and it may be helpful to measure and record the T_{500} time while doing the slump-flow test as a way of conforming uniformity of the SCC from batch to batch.

Table A.2 Viscosity classification and values

Sl. No.	Class	T500 (S)	V-funnel time (S)
1	VS1/VF1	≤ 2	≤ 8
2	VS2/VF2	> 2	9 to 25

VS1/VF1 has good filling ability even with congested reinforcement. It is capable of self –leveling and generally has the best surface finish. However. It is more likely to suffer from bleeding and segregation.

VS2/VF2 has no upper class limit but with increasing flow time it is more likely to exhibit thixotropic effects, which may be helpful in limiting the formwork pressure or improving segregation resistance. Negative effects may be experienced regarding surface finish (blow holes) and sensitivity to stoppages or delays between successive lifts.

Appendix-B

TYPICAL MIX DESIGN PROCEDURE FOR CONCRETE

I. Design stipulations

a) Characteristic compressive strength requires in the field at 28 days	-	$25 N/mm^2$
b) Maximum size of aggregate	-	20mm (Angular)
c) Degree of workability	-	0.9 (Compacting factor)
d) Degree of quality control	-	Good
e) Type of exposure	-	Mild

II. Test data of materials

a) Cement used	-	Ultratech (53 Grade OPC)
b) Properties of cement		
Specific gravity of cement	-	3.14
Standard Consistency	-	33.60%
Initial setting time	-	80mts.
Final setting time	-	240mts.
Strength of mortar cubes (28^{th} day)	-	61.40 MPa

III. Properties of aggregates

a) Coarse aggregate		
Specific gravity	-	2.77
Water absorption	-	0.20%
Free surface moisture	-	Nil
Sieve analysis	-	Graded
Aggregate crushing strength	-	26.00%

b) Fine aggregate

Specific gravity	-	2.67
Water absorption	-	0.9%
Grading	-	Zone II
Surface moisture	-	Nil

IV. Target mean strength

$\bar{f}_{ck}$ = fck+ k×s

Here

fck = 25 MPa, t = 1.65, s = 5.3

$\bar{f}_{ck}$ = 25 + 1.65 ×5.3 = 33.75 MPa

V. Water cement ratio = 0.42

VI. Selection of water and sand content

Water content required per m^3	=	186 kg/m^3
Sand in total aggregate	=	38%

VII. Adjustment in water content and percentage sand

Change in condition	Adjustments required in	
	Water content	Percentage sand in total Aggregate
For zone	0.0%	0.0%
Increase in compaction factor (0.8 - 0.9)	+ 3%	0.0%
Decrease in water cement ratio (0.6 - 0.42)	0.0%	-3.0 %
For Rounded Aggregate	0.0%	3.0%
Total	+3%	-3.0%

Therefore required sand content as percentage of total aggregate by absolute volume = 38 – 3.0 = 35%

Required water content = 186 + [3/100] × 186 = 191.58 lit/m^3

VIII. Determination of cement content

Water Cement ratio	=	0.42
Water	=	191.58 lit/m^3
Cement	=	(191.58/0.42) = 478.95 kg/ m^3
Adopt	=	375 kg/m^3
Hence water	=	157.8 lit/ m^3

IX. Determination of coarse and fine aggregate content.

Percentage air entrapped = 2%

i) For fine Aggregate

$$V = \left(W + \frac{C}{Sc} + \frac{1}{\rho} \times \frac{f_a}{S_{fa}} \right) \times \frac{1}{1000}$$

Where V - Absolute volume of fresh concrete which is equal to gross volume minus volume of entrapped air - 1-0.02 = 0.98

W - Mass of water in kg per cubic meter of concrete =157.80kg/m^3

S_c - Specific gravity of cement = 3.14

ρ - Ratio of fine aggregate to the total aggregate of absolute volume = 0.35

S_{fa} - Specific gravity of fine aggregate = 2.67

S_{ca} - Specific gravity of Coarse Aggregate = 2.77

C - Weight of Cement per m^3 of concrete in kg = 375

f_a - Weight of fine aggregate per m^3 of concrete in kg

$$0.98 = [157.80 + \frac{375}{3.14} + \frac{1}{0.35} \times \frac{fa}{2.67}] \frac{1}{1000}$$

$$f_a = 656.74.68 \text{ kg}$$

ii) For coarse aggregate

c_a - Weight of coarse aggregate per m^3 of concrete in kg

$$0.98 = [157.80 + \frac{375}{3.14} + \frac{1}{0.65} \times \frac{ca}{2.77}] \frac{1}{1000}$$

$$c_a = 1265.34 \text{ kg}$$

X. Admixture

Dosage of admixture as per suppliers data After adjustments for water absorption the final mix proportion is obtained as

Table B.1 Materials required for 1m^3 of control concrete

Material	Cement (kg)	Fine aggregate (kg)	Coarse aggregate (kg)	Water (L)	Super plasticizer (L)
Quantity	375	639	1288	157.8	2.25

Appendix-C

DATA USED FOR DEVELOPING MIX DESIGN METHODOLOGY FOR LSCC

C1 General

This appendix provides the details of data used for developing mix design methodology for LSCC. Table C.1 presents the proportions ingredients arrived at for M20 to M40 mixes based on trail mixes. Figures C.1 to C.3 shows the variation of different ingredients (cement, fly ash and weathered laterite aggregate) with the grade of laterised self compacting concrete (LSCC).

Table C.1 Quantities of materials for laterised self compacting concrete with fly ash as addition

Sl. No	Grade of LSCC	Cement content (kg/m^3)	Fly ash content (kg/m^3)	Laterite all-in aggregate content (kg/m^3)	Water content (kg/m^3)	Superplasticiser content
1	M20	338.6	336.6	1483.4	236.0	6.8
2	M20	372.8	369.8	1420.0	271.3	7.5
3	M20	399.7	387.6	1369.4	236.0	8.0
4	M20	429.6	390.0	1346.7	242.7	8.2
5	M20	432.3	376.9	1348.9	243.0	8.1

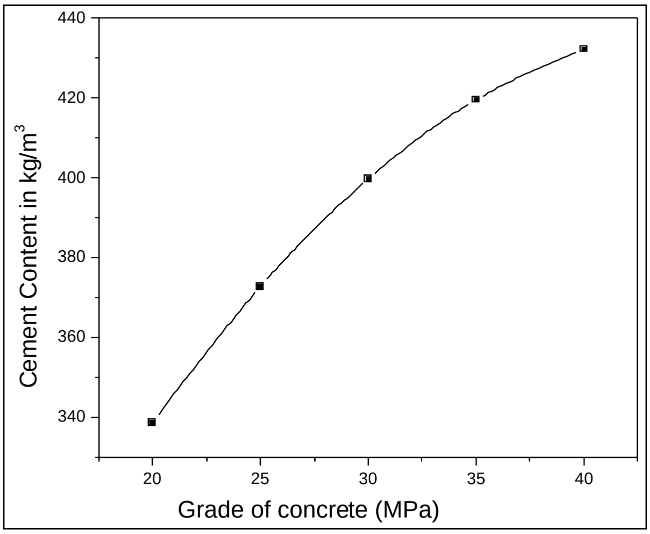

Figure C.1 Relation between cement content and grade of LSCC.

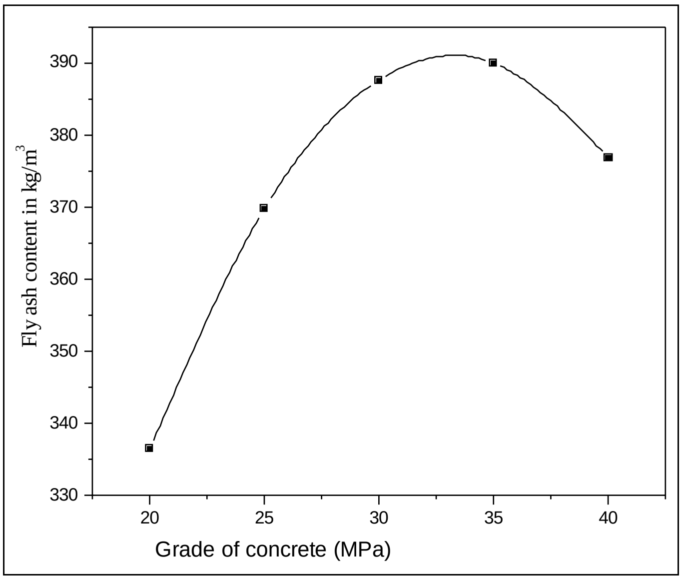

Figure C.2 Relation between fly ash content and grade of concrete.

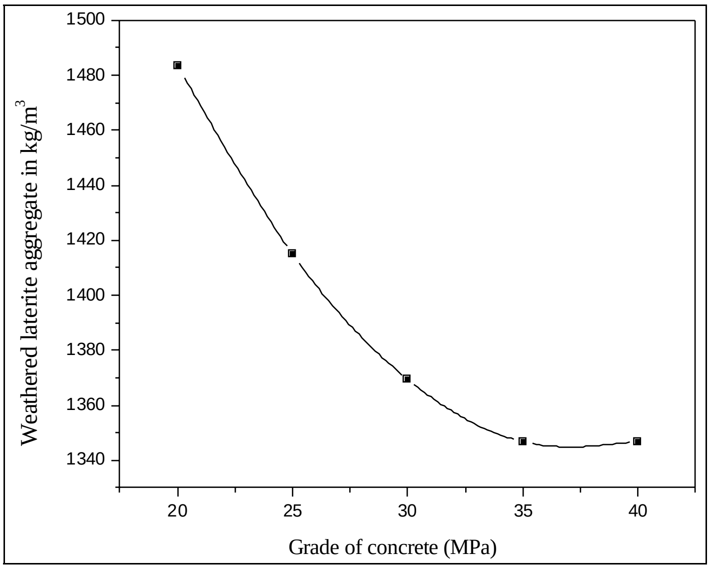

Figure C.3 Relation b/w weathered laterite aggregate and grade of concrete.

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