DURABILITY PROPERTIES
OF
HIGH PERFORMANCE FIBER REINFORCED CONCRETE

A dissertation submitted
In partial fulfilment of the requirements for
For the award of degree of

MASTER OF ENGINEERING
IN
CIVIL (STRUCTURES) ENGINEERING

Submitted by

SHIVINDER SINGH
(ROLL NO. 821122010)

Under the guidance of

Dr. PREM PAL BANSAL
Associate Professor

CIVIL ENGINEERING DEPARTMENT
THAPAR UNIVERSITY, PATIALA- 147004
OCTOBER 2015

CERTIFICATE

I Shivinder Singh hereby declare that this thesis entitled, **"Durability Properties of High Performance Fiber Reinforced Concrete"** is an authentic record of my own review carried out as requirements for the award of degree **Master of Engineering (Structures)** in the **Civil Engineering Department, Thapar University, Patiala**, under the supervision and guidance of **Dr. Prem Pal Bansal Associate Professor, Department of Civil Engineering, Thapar University, Patiala**. The matter embodied in this thesis has not been submitted in part or full to any other University or Institute for the award of any degree.

Shivinder Singh

Date: 05/10/2015
Place: Patiala

Shivinder Singh
Regd. No. (821122010)

It is certify that the above statement made by the student concerned is correct to best of my knowledge and belief.

(Dr. Prem Pal Bansal)
Associate Professor
CED, Thapar University
Patiala-147004

Countersigned by:

Dr. Naveen Kawatra
Professor & Head of Department,
Department of Civil Engineering,
Thapar University, Patiala.

Dr. S. S. Bhatia
Dean of Academic Affairs,
Thapar University,
Patiala.

ACKNOWLEDGEMENT

I express my deep gratitude and respects to my supervisor **Dr. Prem Pal Bansal, Associate Professor in Civil Engineering Department,** for their keen interest and valuable guidance, motivation and constant encouragement during the course of work. I thank him for their great patience, constructive criticism and myriad useful suggestions apart from in valuable guidance to me. My first and foremost offering of thanks goes to the architect who shaped my dreams into reality.

I would like to convey my sincere gratitude to my friends, colleagues, my wife and loving son for their support, co-operation and their timely help and valuable discussions.

I owe my sincere thanks to all the staff members of **Civil Engineering Department** for their support and encouragement. The meaning of my life and work is incomplete without paying regards to my respected parents whose blessings and continuous encouragement have shown me the path to achieve the goals.

And above all, I pay my regards to the **Almighty** for his love and blessings.

Shivinder Singh

Shivinder Singh
Regd. No. 821122010

ABSTRACT

The term High Performance Fiber Reinforced Concrete (HPFRC) refers to a relatively new class of advanced cementitious composite materials whose mechanical and durability properties far superior than those of conventional concrete. This class of concrete has been demonstrated to facilitate solutions that address specific problems. Initial material development research on HPFRC began more than two decades ago. First structural deployments began in the late 1990s. HPFRC-class materials are defined as cementitious-based composite materials with discontinuous fiber reinforcement that exhibit compressive strength above 21.7 ksi (150 MPa), pre- and post-cracking tensile strength above 0.72 ksi (5 MPa), and enhanced durability via a discontinuous pore structure.

HPFRC material has unique combination of superior properties and design flexibility with advantages such as speed of construction improved aesthetics, superior durability and impermeability against corrosion, abrasion and impact which translates to reduced maintenance and a longer life span for the structure.

One of the major disadvantages in High Performance Fiber Reinforced Concrete (HPFRC) is its high ordinary Portland cement (OPC) content, which directly translates into an increase in OPC production. More OPC production results in increased emission of greenhouse gases, as well increased electrical energy consumption and concrete price.

This thesis aims to find the durability properties like compressive strength, tensile strength, flexural strength and permeability (with RCPT and Sorptivity test) of HPFRC with potable water and sea water curing conditions. A comprehensive literature review is carried out for better understanding of durability properties of HPFRC .Compiled from many references representing research, development, and deployment efforts around the world, this thesis report provides a framework for gaining a deeper understanding of HPFRC.

CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1

INTRODUCTION

1.1 INTRODUCTION

Since ancient time, mankind has been searching for construction materials with higher and higher performance so they can build taller, longer and better structures. They probably started with mud, straw and wood because they were handily available. Mud brick and adobe structures are still being built by native people in many parts of the world. They can actually last a long time, given that the weather is favorably dry in these areas.

Fig. 1.1: Greek Parthenon

The Minoan started to use lime mortar around 2000 BC. The Greek used it too. But this material can dissolve itself in water and is therefore not very weather resistant. The Romans made a significant improvement on it by adding a volcanic ash from the town of Pozzuoli (hence the name pozzolana). When tools became available, stone was widely used by the Greeks, Egyptians and Romans, **Fig. 1.1.**

Fig.1.2: Roman Pantheon

The concrete at that time, a mixture of lime, sand, stone and water, is pretty much the same as that used in many areas of the ancient world, or even today in many under-developed countries. The high compressive strength of the ancient cement, in combination with brick and stone, allowed them to build large arches and great domes. A stone girder can hardly span 5 meters. An arch, on the contrary, can span over 50 meters. The Pantheon **(Fig. 1.2)** in Rome, built around 128 AD, has a dome that spans 43.3 meters, with stones and Roman concrete, which was the largest dome in the world for almost 1900 years.

Portland cement was officially introduced by Josepf Aspdin in 1824 and reinforced concrete was first patented by W.B. Wilkinson in 1854. Accordingly, the history of reinforced concrete is only about 150 years.

Interrelationship between a material and structural form there is a close relationship between the construction material available and the type of structures we build.

Obviously, certain types of structures can only be built after we have developed the appropriate material for construction

Today, we are basically utilizing wood, concrete and steel in practically all of our construction. Composites and fiber-reinforced polymers are rather new introductions, they have yet to establish their empire of applications.

Similarly, ultra high performance concrete with strength in the range of 150 MPa is a new material. It is not really concrete anymore. It also has to establish its empire of applications.

1.2 BACKGROUND

In Canada, the first UHPC bridge was constructed in 1997. This pedestrian bridge consists of a precast, post-tensioned space truss. At least 26 bridges have been built in Canada using UHPC in one or more components.

Ultra-high performance concrete (UHPC) in its present form became commercially available in the United States in about 2000.The Federal Highway Administration (FHWA) began investigating the use of UHPC for highway infrastructure in 2001 and has been working with State transportation departments to deploy the technology since 2002. This work has led to the use of UHPC in several bridge applications, including precast, pre stressed girders; precast waffle panels for bridge decks; and as a jointing material between precast concrete deck panels and girders and between the flanges of adjacent girders. At the same time, research work has been underway at several universities in the United States.

In 2002, the first recommendations on the use of UHPC in structures were published in France. This initial document addressed mechanical properties, structural design, and durability. Since 2002, several buildings & bridges have been built in France using UHPC. In 2009, several papers published in French recommended updates to the recommendations. A similar set of design recommendations was developed for use in Japan.

In Germany, a 12 million euro research program, begun in 2005, has just been completed. That program, funded by the German Research Foundation, involved 34 research projects at more than 20 research institutes in Germany. The purpose of the program was to elaborate on the basic knowledge so that reliable technical standards could be developed. The goal was to make UHPC a reliable, commonly available, economically feasible, regularly applied material. Several bridges that use UHPC have been built in Germany.

Other countries with bridges using UHPC include Australia, Austria, Croatia, Italy, Japan, Malaysia, the Netherlands, New Zealand, Slovenia, South Korea, and Switzerland. The literature search identified more than 90 completed bridges using UHPC in one or more components. A major research program is currently underway in South Korea to investigate the use of UHPC in cable-stayed bridges. It is obvious therefore, that UHPC is receiving worldwide attention.

The evolution of UHPC into its present formulation has been a gradual process occurring over many years. Several papers have summarized this development. Researchers identified many of the significant advances in the technology over the last 5 decades.

1.3 DEFINITION OF ULTRA HIGH PERFORMANCE CONCRETE

A performance enhanced concrete or ultra high performance concrete (UHPC) is a specialized series of concrete designed to provide several benefits in the construction of concrete structures that cannot always be achieved routinely using conventional ingredients, normal mixing and curing practices. In the other words, a high performance concrete is a concrete in which certain characteristics are developed for a particular application and environment, so that it will give excellent performance in the structure in which it will be placed, in the environment to which it will be exposed, and with the loads to which it will be subjected during its design life. It includes concrete that provides either substantially improved resistance to environmental influences (durability in service) or substantially increased structural capacity while maintaining adequate durability. It may also include concrete, which significantly reduces construction time without compromising long-term serviceability. It is, therefore, not possible to provide a unique definition of UHPC without considering the performance requirements of the intended use of the concrete. Examples of characteristics that may be considered critical in an application requiring performance enhancement are ease of placement and compaction without segregation, early-age strength, long-term mechanical properties, permeability, density, heat of hydration, toughness, volume stability, and long life in severe environments, i.e. durability. Concretes possessing many of these characteristics often achieve higher strength, Therefore, UHPC is often of ultra-high strength, but ultra-high strength concrete may not necessarily be of high performance. Thus, in practical applications of this type of concrete, the emphasis has in many cases gradually shifted from the compressive strength to other properties of the material, such as a high modulus of elasticity, high density, low permeability, and high resistance to same forms of attacks. The cost and other benefits derived may include less material, light and fewer structural elements, reduced maintenance, extended life cycle and aesthetics.

Construction materials have different designations in different countries. In reality, today's world is rather small. We are using basically very similar materials in various parts of the world even though they may have different names.

Literally, the German term "Ultrahochfester Beton" is not really the same as "Ultra High Performance Concrete." "Hochfest" describes the internal condition of the concrete while "High Performance" refers to the external characteristics of the material. However, we are using them interchangeably.

In the United States, the definition of Ultra High Performance Concrete by the Federal Highway Administration includes **eight performance** characteristics:

- Freeze-thaw durability
- Scaling resistance
- Abrasion resistance
- Chloride penetration
- Compressive strength

- Modulus of elasticity
- Shrinkage
- Creep

1.4 MAIN INGREDIENTS OF UHPC

Following are the main ingredients of UHPC:

1.4.1 Portland Cement

The raw materials required for manufacture of Portland cement are calcareous materials, such as limestone or chalk and argillaceous material such as shale or clay. Cement factories are established where these raw materials are available in plenty. Cement factories have come up in many regions in India, eliminating the inconvenience of long distance transportation of raw and finished materials.

The process of manufacture of cement of grinding the raw materials, mixing them intimately in certain properties depending upon their purity and composition and burning them in a kiln at a temperature of about 1300 of 1500^0 C, at which temperature, the material sinters and partially fuses to form nodular shaped clinker. The clinker is cooled and ground to fine power with addition of about 3 to 5% of gypsum. The product formed by using this procedure is Portland cement.

There are two processes known as “wet” and “dry” processes depending upon whether the mixing and grinding of raw materials.

For many years the wet process remained popular because of the possibility of more accurate control in the mixing of raw materials. The technique of intimate mixing of raw materials in powder form was not available then. Later, the dry process gained momentum with the modern development of the technique of dry mixing of powdered materials using compressed air. The dry process requires much less fuel as the materials are already in a dry state.

1.4.2 Silica Fume

Silica fume, also called micro silica, is a light to dark grey or pink white cementing material composed of at least 85 percent ultra-fine, amorphous non-crystalline (glassy) spherical silicon dioxide (SiO_2) particles. It is produced as a by-product during the manufacture of silicon metal or ferrosilicon alloys by reduction of high purity quartz in a submerged-arc electric furnace heated to 2000^0 C with coal coke and wood chips as fuel. The individual particles are extremely fine, approximately 1/50th the size of an average Portland cement particle (0.1 to 0.3 µm) with minimum specific surface area of 15000 m^2/kg. As in the case of other pozzolanas the efficiency of silica fume depends upon its mineralogy and particle size distribution. The extremely fine particle size, large surface area and high content of highly reactive amorphous silicon dioxide give silica fume the super pozzolanic properties. This material, which is considered to be one of the state-of-the-art materials, has been also described as an assessed as a partial replacement for Portland cement. Silica fume is not a chemical special chemical action distinct from pozzolanas. There are various advantages in using silica fume such as reduction in bleeding and segregation of fresh concrete, and improvements in the strength and durability characteristic of hardened concrete. The combination of high reactivity and extreme fineness results in the possibility of producing more dense concrete with a very low porosity, the pores being small and discontinuous, and therefore, with a high strength and a low penetrability.

1.4.3 Aggregates

Generally 85% of the volume of concrete is occupied by the aggregates. Aggregates are those inert materials, which when bound together by cement, form mortar or concrete Aggregates are known as inert materials because they remain chemically inactive during setting and hardening of cement. Aggregates mostly used for the manufacture of concrete are sand, blast furnace slag, saw dust, crushed rock etc.

The aggregates are cheaper than cement and a concrete mix which makes use of maximum quantity of aggregates is economical. Aggregates provide greater volume stability and better durability to concrete.

According to size the aggregates are classified into the following **three types**:

a) Fine Aggregates. The aggregates which pass through 4.75 mm sieve and retained on 75 micron (0.075 mm) sieve are known as fine aggregates.

The lower size limit of sand is about 0.07 mm. The material having particle size from 0.06 mm to 0.002 mm is classified as silt and those with particle size less than 0.002 mm is known as clay. The soft deposit consisting of sand, silt and clay in about equal proportions is called loam.

Fine aggregates fill up the voids in coarse aggregates and thus strong concrete with less quantity of cement is obtained by using fine aggregates. It reduces the cost of concrete. It also increases workability of concrete.

b) Coarse Aggregates. The aggregates which pass through 75 mm sieve and retained on 4.75 mm sieve are known as coarse aggregates.

The maximum particle size of coarse aggregate is 75 mm and its minimum particle size is 4.75 mm the aggregate of particle size greater than 75 mm is known as **cyclopean aggregate.** The size of coarse aggregates depends on the type of work and arrangement of reinforcement provided in the concrete.

c) All-In-Aggregates. The aggregate which is the combination of coarse aggregate and fine aggregate is called all-in-aggregate. These aggregates are not generally used for making high quality concrete.

1.4.4 Water

Water is the most important and least expensive ingredient of concrete. A part of mixing water is utilized in the hydration of cement to from the binding matrix in which the inert aggregates are held in suspension until the matrix has hardened. The remaining water serves as lubricant between the fine and coarse aggregates and makes concrete workable, i.e. readily place able in forms.

The strength and durability of concrete is reduced due to the presence of impurities in the mixing water. The effects are expressed mainly in terms of differences in the setting times of concrete mixes containing proposed mixing water as compared in distilled water and concrete strengths compared with those of control specimens prepared with distilled water. The differences in 28 days compressive strength up to 10 percent of control test in generally considered being a satisfactory measure of the quality of mixing water.

The water for the mixing and curing of concrete should be free form injurious amounts of deleterious materials. The unwanted situations, leading to the distress of concrete, have been found to be a result of, among others, the mixing and curing water being inappropriate quality. Potable water is generally considered satisfactory for mixing concrete. In the case of doubt about the suitability of water, particularly in remote areas or where water is derived from sources not normally utilized for domestic purposes, water should be tested.

1.4.5 Super Plasticizers (High Range Of Water Reducers)

Super plasticizers constitute a relatively new category and improved version of plasticizer, the use of which was developed in Japan and Germany during 1960 and 1970 respectively. They are chemically different from normal plasticizers. Use of super plasticizers permit the reduction of water to the extent up to 30 per cent without reducing workability in contrast to the possible reduction up to 15 per cent in case of plasticizers.

The use of super plasticizer is practiced for production of flowing, self-leveling, and self-compacting and for the production of high strength and high performance concrete.

The mechanism of action of super plasticizers is more or less same as in case of ordinary plasticizer. Only thing is that the super plasticizers are more powerful as dispersing agents and they are high range water reducers. They are called High Range Water Reducers in American literature. It is the use of super plasticizer which has made it possible to use w/c as low as 0.25 or even lower and yet to make flowing concrete to obtain strength of the order 120 MPa or more. It is the use of super plasticizer which has made it possible to use fly ash, slag and particularly silica fume to make high performance concrete.

The use of super plasticizer in concrete is an important milestone in the advancement of concrete technology. Since their introduction in the early 1960 in Japan and in the early 1970 in Germany, it is widely used all over the world. India is catching up with the use of super plasticizer in the construction of high rise buildings, long span bridges and the recently become popular **'Ready Mixed Concrete Industry'.**

1.4.5 a) Benefits of Super plasticizers

- At the same w/c ratio makes more workable concrete than conventional concrete.
- For the same workability, it permits the use of lower w/c ratio.
- As a consequence of increased strength with lower w/c ratio, it also permits a reduction of cement content.
- Super plasticizers produce a homogeneous, cohesive concrete generally without any tendency for segregation and bleeding.

1.4.5 b) Classification of Super plasticizer

Following are few polymers which are commonly used as base for super plasticizers.

- Sulphonated malanie-formaldehyde condensates (SMF).
- Sulphonated naphthalene-formaldehyde condensates (SNF).
- Modified lignosulphonates (MLS).
- Acrylic polymer based (AP)
- Copolymer of carboxylic acrylic acid with acrylic ester (CAE)
- Cross linked acrylic polymer(CLAP)
- Polycarboxylate ester (PC)
- Multicarboxylatethers (MCE)
- Combinations of above.

As far as our country is concerned, at present we manufacture and use the first four types of super plasticizers. The new generation super plasticizers have been tried in recent projects, but it was not found feasible for general usage on account of high cost. The first four categories of products differ from one another because of the base component or on account of different molecular weight. As a consequence each commercial product will have different action on cements. Whilst the dosage of conventional plasticizers do not exceed 0.25% by weight of cement in case of lignosulphonates, or 0.1% in case of carboxylic acids, the products of type SMF or NSF are used considerably high dosage (0.5% to 3.00%), since they do not entrain air. The modified lignosulphonate (LS) based admixtures, which have an effective fluizing action, but at the relatively high dosage, they can produce undesirable effects, such as accelerations or delay in setting times. Moreover, they increase the air-entrainment in concrete.

Plasticizers and super plasticizers are water based. The solid contents can vary to any extent in the products manufactured by different companies. Cost should be based on efficiencies and solid content, but not on volume or weight basis. Generally in projects cost of super plasticizers should be worked for one cubic meter of concrete. Consistency in the quality of super plasticizers supplied over a period of time can be tested and compared by "Infrared Spectrometry".

1.4.5 c) Effects of Super Plasticizers on Fresh Concrete

It is to be noted that dramatic improvement in workability is not showing up when plasticizers or super plasticizers are added to very stiff or what is called zero slump concrete at nominal dosages. A mix with an initial slump of about 2 to 3 cm can only be fluidized by plasticizers or super plasticizers at nominal dosages. A high dosage is required to fluidity no slump concrete. An improvement in slump value can be obtained to the extent of 25 cm or more depending upon the initial slump of the mix, the dosage and cement content. It is often noticed that slump increases with increase in dosage. But there is no appreciable increase in slump beyond certain limit of dosage. As a matter of fact, the over dosage may sometime harm the concrete.

1.5 PERFORMANCE CRITERIA FOR STRUCTURES

For today's structures, we look for materials with four distinctive properties strength, workability, durability and affordability. The first three properties basically include all the eight performance requirements listed in article 1.3 and affordability is cost.

When we say high performance, we refer to the improvement in some or all of these properties. Sometimes, we have to give up a little in one to gain a little in the other.

But, in general, with time, all these properties improve. We will discuss these four properties one by one as follows:

1.5.1 Strength

Higher strength offers savings in material. Weight, or the structural dead load, is a major loading in the design of structures. Consequently, higher strength usually gives us two advantages: less material and less weight. The reduction in weight in turn reduces the demand on material because it reduces the load the structure has to carry with strength of 150 MPa, the UHPC is almost like steel except its tensile capacity is still comparatively low so it cannot be used like steel. But this is many folds higher than the regular concrete, the strength of which is around 50 MPa.

1.5.2 Workability

A structure is not only designed, but it also must be constructed. Workability affects the cost and the time required to build the structure. Obviously time and cost are often the two fundamental determinants on whether a bridge or a certain type of structure will be built.

Workability also implies that a material is reliable and consistent. We need better equipment to achieve that in an industrial scale. A new material, such as UHPC, is inherently disadvantaged because construction methods need time to be developed.

1.5.3 Durability

When we look at some of the ancient structures of Roman and Byzantine eras that are still standing, we wonder how long our structures will last. Our ancient fellow engineers just built major structures based on their best knowledge and usually expected the structures to last forever. Today, we know that nothing will last forever and we become more humble and design our buildings and bridges to a defined design life. With the design life of major bridges usually being 100 to 150 years, we are in need of durable materials that will last a long time and are easy to maintain. UHPC does offer us good potential in this respect. However, in an engineering world that values performance records, a certain amount of time will be needed to assure people that the long term performance of the material is what the laboratory tests have shown us.

1.5.4 Affordability

Cost is often a determining factor on whether a structure will be built. There are probably other good construction materials that can be used for construction except that their high cost may have prevented them from being used.

A potentially good but expensive material may become affordable when its application is more widespread due to mass production, while its application can only get widespread if its cost is sufficiently low.

For example, Stainless steel is good for many applications in construction. However, the higher price of stainless steel is a hindrance to its widespread use.

1.6 UHPC TODAY

Obviously, “high” is a relative term. The term “ultra-high” is more so. The Laurentienne Building in Montreal, built in 1984, used a 106 MPa high performance concrete and the Two Union Square Building in Seattle, USA, built in 1988, utilized a 145 MPa high performance concrete. Going to a UHPC with strength higher than 150 MPa certainly is another big step forward.

Various tests have confirmed UHPC’s performance in the laboratories. They show high strength and durability. Theoretically, we are able to use it for daily applications, wherever high strength and durability are beneficial. The basic principle is to use stronger aggregates, micro silica, and water reducing agents to raise its strength, steel fibers to prevent brittle failure and polypropylene fibers to increase its fire resistance, and so forth. In general, with steam curing, we are able to reach strength in the range of 200 MPa or higher. The resulting concrete basically meets all eight performance criteria of the FHWA for high performance concrete. The material itself is therefore available. The problem is the ease of application and the price.

1.7 TERMINOLOGY

Various terms are used to refer to cementitious-based composite materials with high compressive strength and enhanced durability. These include the following:

- Compact reinforced composite (CRC).
- Densified small-particle (DSP) concrete.
- Fiber-reinforced high-performance concrete (FRHPC).
- High-performance fiber reinforced cement composite (HPFRCC).
- Macro defect free (MDF) concrete.
- Multi-scale fiber-reinforced concrete (MSFRC).
- Reactive powder concrete (RPC).
- Steel fibrous cement-based composite (SFCBC).
- Ultra-high performance concrete (UHPC).
- Ultra-high performance fiber-reinforced cementitious composite (UHPFRCC).
- Ultra-high performance fiber-reinforced concrete (UHPFRC).
- Ultra-high strength concrete (UHSC).
- Ultra-high strength cement-based composite.
- Ultra-high strength cementitious material.
- Ultra-high strength fiber-reinforced cementitious composite.

In addition, various patterns of hyphens are used to form compound adjectives. For this report, the product is generally called ultra-high performance concrete or UHPC unless it is necessary to differentiate the different types. Descriptions of some of the different types are provided by **Rossi (2001)**. Although calling the different types by a single name may not be technically correct, it simplifies understanding the available information.

1.8 COSTS

The initial unit quantity cost of UHPC far exceeds that of conventional concrete. Consequently, applications have focused on optimizing its use by reducing concrete member thickness, changing concrete structural shapes, or developing solutions that address shortcomings with existing non-concrete structural materials. UHPC is a very durable product, and structures that use it are expected to have a longer service life and require less maintenance than structures built with conventional concrete.

Piotrowski and Schmidt (2012) conducted a life cycle cost analysis of two replacement methods for the Eder bridge in Felsberg, Germany. One method used precast UHPC box girders filled with lightweight concrete. The second method used conventional prestressed concrete bridge members. Although the UHPC had higher initial costs, the authors predicted the life cycle cost over 100 years would be less for the UHPC Bridge.

1.9 NEED OF HOUR

To encourage greater implementation of UHPC in the infrastructure projects. The following activities and documents are needed in approximate order of priority:

- Studies showing the cost effectiveness of UHPC in various applications.
- Design and construction guide specifications for structures made with UHPC.
- Research to address some of the missing information needed in the structural design guidelines.
- Standard test methods and material specifications for UHPC.
- Production procedures for precast and cast-in-situ construction.
- Broader geographic distribution of demonstration projects.
- Ongoing and greater distribution of technical information.

1.10 ADVANTAGES OF HIGH PERFORMANCE FIBER REINFORCED CONCRETE

- Used for isolation and containment of nuclear wastes.
- High tensile, flexural, and compressive strengths.
- Good long-term durability with respect to freeze and thaw cycles.
- Low permeability to water and aggressive solutions.
- Good chemical resistance.
- Good resistance against corrosion.

- Self-placing, requires no internal vibration, the large amount of super plasticizers still makes it workable.
- Basically, structures needing light and thin components and anything that needs extra safety or security such as blast resistant structures.
- Superior strength combined with higher shear capacity result in significant dead load reduction.
- Improved seismic performance by reducing inertia load with lighter member allowing larger deflections with reduced cross sections and providing higher energy absorption.
- Low and non-interconnected porosity diminishes transfer, making penetration of liquid/gas non-existent.
- Absolutely weather-proof, resistant to frost.

1.11 DISADVANTAGES OF HIGH PERFORMANCE FIBER REINFORCED CONCRETE

- The high material cost, high energy consumption and embedded CO_2 for HPFRC are the typical disadvantages that restrict its wider application.
- There is no code for the design criteria of HPFRC.
- HPFRC replace steel in compression members where durability issues risk. So long term properties are not yet known.
- The least costly components of conventional concrete are eliminated or replaced by more expensive elements.
- The mineral component optimization alone results in a substantial increase in cost over and above that of conventional concrete (5 to 10 times higher than conventional concrete).
- Since HPFRC is in its infancy the long term properties are not yet known.

1.12 AIM AND OBJECTIVES OF THESIS

The review of literature reported in Chapter 2 indicates that scanty work has been carried out to investigate the durability properties high performance fiber reinforced concrete. Therefore, the main objective of the present study is to investigate the durability properties of high performance fiber reinforced concrete with potable and sea water curing condition.

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

Ultra High Performance Concrete (UHPC) with a high compressive strength of more than 150 MPa and an improved durability marks a quantum leap in concrete technology. This high performance material offers a variety of interesting applications. It allows the construction of sustainable and economic buildings with an extraordinary slim design. Its high strength and ductility makes it the ultimate building material e.g. for bridge decks, storage halls, thin-wall shell structures and highly loaded columns.

Beside its improved strength properties, its outstanding resistance against all kinds of corrosion is an additional milestone on the way towards no-maintenance constructions.

UHPC has very special properties that are remarkably different to the properties of normal and high performance concrete. For complete utilization of UHPC’s superior properties, special knowledge is required for production, construction and design.

Worldwide this material is under detailed exploration. Several constructions or structural elements were already built utilizing UHPC.Various researches have worked on different aspects of ultra high performance concrete starting from its proper preparation with different materials to studying its physical and chemical properties. A brief review of the work carried out in the subject area is presented in subsequent sections.

Aldahdooh, M.A.A et al (2013) represents one of the major disadvantages in ultra-high-performance-fiber reinforced concrete (UHP-FRC) is its high ordinary Portland cement (OPC) content, which directly translates into an increase in OPC production. More OPC production results in increased emission of greenhouse gases, as well increased electrical energy consumption and concrete price. This study is aimed at adjusting the binder content (OPC and silica fume (SF) contents) of UHP-FRC using the response surface method. The present investigation shows that, for a given water/binder and super plasticizer/OPC, the compressive strength are independent of the binder content, whereas the flow depends on the binder content. Increasing the binder content does not enhance the strength compared with the required design strength because the capillary porosity increases with increasing OPC content; however, the workability increases. The final result is the production of a UHP-FRC with an OPC content of 720.49 kg/m^3, an SF content of 214.25 kg/m^3, a compressive strength of 181.41 MPa, a direct tensile strength of 12.49 MPa, a bending tensile strength of 30.31 MPa, and a flow of 167 mm.

2.2 CONSTITUENT MATERIALS AND MIX PROPORTIONS

UHPC formulations often consist of a combination of portland cement, fine sand, silica fume, high-range water-reducing admixture (HRWR), fibers (usually steel), and water. Small aggregates are sometimes used, as well as a variety of chemical admixtures. Different combinations of these materials may be used, depending on the application and supplier.

According to **Graybeal (2006)** the UHPC used most often in North America for both research and applications is a commercial product known as Ductal®. **Table 2.1** shows a typical composition of this material.

Table 2.1: Typical composition of UHPC Ductal®

S. No.	**Material**	**lb/yd^3**	**kg/m^3**	**Percentage by Weight**
1	Portland Cement	1,200	712	28.5
2	Fine Sand	1,720	1,020	40.8
3	Silica Fume	390	231	9.3
4	Ground Quartz	355	211	8.4
5	HRWR	51.8	30.7	1.2
6	Accelerator	50.5	30.0	1.2
7	Water	184	109	4.4
8	Steel Fibers	263	156	6.2

The following recommendations for mix proportions were developed for use with commercially available constituent materials:

- Cement with a moderate fineness and C_3A content significantly lower than 8 percent.
- Sand-to-cement ratio of 1.4 for a maximum grain size of 0.8 mm (0.03 inches).
- Silica fume with very low carbon content at 25 percent of the weight of cement.
- Glass powder with a median particle size of 67 x 10^{-6} inches (1.7 µm) at 25 percent of the weight of cement.
- High-range water-reducing admixture.
- Water-cement ratio of about 0.22.
- Steel fibers at 2.5 percent by volume.

According to **Wille.k et al. (2006)** by optimizing the cementations matrix for compressive strength, packing density, and flow ability using very high strength, fine-diameter steel fibers and tailoring the mechanical bond between the steel fiber and cement matrix, 28-day compressive strengths in excess of 30 ksi (200 MPa) on 2-inch (50-mm) cubes were achieved with no heat or pressure curing. In addition, a tensile strength of 5.0 ksi (34.6 MPa) at a strain of 0.46 percent was obtained. The UHPC incorporated materials available in the United States and was mixed in a conventional concrete mixer. **Table 2.2** gives one mix proportion.

Table 2.2: UHPC mix proportions of CRC by weight

Sr. No	Material	Proportions
1.	Portland Cement	1.0
2.	Fine Sand1	0.92
3.	Silica Fume	0.25
4.	Glass Powder	0.25
5.	HRWR	0.0108
6.	Steel Fibers	0.22 to 0.31
7.	Water	0.18 to 0.20

Maximum size of 0.008 inches (0.2 mm)

Habel et al. (2008) reported that it is possible to produce self-consolidating UHPC for use in precast products and cast-in-place (CIP) applications without requiring heat or pressure treatment during curing. This mix design was further developed and implemented in a research program conducted by **Kazemi and Lubell (2012).**

Holschemacher and Weißl (2005) investigated different mix proportions to minimize material costs without sacrificing the beneficial properties of UHPC. Through careful selection of aggregates, cement type, cementations materials, inert filler, and HRWR, it was possible to produce UHPC with good workability and moderate material costs.

The concept of combining different size molecular admixtures to facilitate UHPC dispersion was studied by **Plank et al. (2009)**. The possibility of replacing silica fume in UHPC with metakaolin, pulverized fly ash, limestone micro filler, siliceous micro filler, micronized phonolith, or rice husk ash has been investigated. The use of local materials rather than proprietary products has also been pursued.

Schmidt et al. (2007) reported two mix proportions for a bridge in Germany. The first mix contained 1,854 lb/yd^3 (1,100 kg/m^3) of cement, 26-percent silica fume as a percentage of the cement content, quartz sand, 6 percent steel fibers by volume, HRWR, and a water-binder ratio of 0.14.The second mix contained 2,422 lb/yd^3 (1,437 kg/m^3) of cement and 9-percent steel wool and steel fibers combined.

Collepardi et al. (1997) reported that the replacement of fine ground quartz sand with an equal volume of well-graded natural aggregate with a maximum size of 0.3 inches (8 mm) did not change the compressive strength at the same water-cement ratio.

Coppola et al. (1997) investigated the influence of high-range water-reducing admixture type on the compressive strength. They reported that acrylic polymer admixtures allowed the use of lower water-cement ratios and resulted in higher compressive strengths compared with naphthalene and melamine admixtures.

Researchers led by **Rossi,P. et al. (2005)** at the Laboratory Central des Ponts et Chaussees (LCPC) in Paris developed a UHPC-class material referred to as CEMTEC. The proportions of this UHPC are presented in **table 2.3**.

Table 2.3: UHPC mix proportions for CEMTEC

Sr. No	**Material**	**Ib/yd^3**	**kg/m^3**
1.	Portland Cement	1,770	1,050
2.	Sand	866	514
3.	Silica Fume	451	268
4.	HRWR	74	44
5.	Steel Fibers	1,446	858
6.	Water	303	180

In a study of the durability of UHPC, **Teichmann and Schmidt (2002)** used the mix proportions shown in **table2.4**. Mix 1 had a maximum aggregate size of 0.32 inches (8 mm) provided by the sand. Mix 2 had a maximum aggregate size of 0.32 inches (8 mm) provided by the basalt.

Table2.4: UHPC mix proportions from Teichmann and Schmidt

		Mix 1		Mix 2	
Sr. No	**Material**	**Ib/yd^3**	**kg/m^3**	**Ib/yd^3**	**kg/m^3**
1.	Cement	1,235	733	978	580
2.	Silica Powder	388	230	298	177
3.	Fine Quartz 1	308	183	503	131
4.	Fine Quartz 2	0	0	848	325
5.	HRWR	55.5	32.9	56.2	33.4
6.	Sand	1,699	1,008	597	354
7.	Basalt	0	0	1,198	711
8.	Steel Fibers	327	194	324	192
9.	Water	271	161	238	141
10.	Water-Binder Ratio	0.19	0.19	0.21	0.21

2.3 MECHANICAL PROPERTIES OF ULTRA HIGH PERFORMANCE CONCRETE

Enhancement of mechanical properties of concrete for higher strength and ductility has received a great deal of attention in recent years in the context of building a sustainable society. A variety of new types of concrete, such as high performance/strength concrete and steel/polymer Fiber Reinforced Concrete (FRC), have been invented for such purpose. In recent years, a new kind of fiber reinforced concrete, called Ultra High Performance Fiber Reinforced Concrete (UHPFRC) has been invented. It is characterized by high ratio (up to 10%) of steel fibers of diameter 0.15–0.20 mm, and high tensile strength in the range of 859–2000 MPa . It often has cement content as high as 900–1000 Kg/m^3. Very fine sand or quartz powder and silica fume are also essential to achieve high particle packing density.

It is important to note that the dispersion and orientation of the fiber reinforcement are critical parameters that influence the mechanical behavior of UHPC. The fiber enforcement serves to resist tensile stresses in the UHPC component both before and after tensile cracking of the UHPC matrix. Post-cracking mechanical response of UHPC is particularly susceptible to degradation from disadvantageous fiber dispersion and/or orientation. Mixing and placing methods can affect the hardened UHPC mechanical response and thus must be appropriately coordinated to ensure acceptable mechanical performance.

Feng J. et al. (2014) studied, a practical model to simulate the pullout performance of hooked steel fiber in ultra-high-performance concrete is proposed. Straight and hooked fiber pullout tests were performed to evaluate the pullout mechanism, based on which slip hardening, matrix spelling and tunnel damage assumptions are made. With energy conservation, static and fracture mechanical analyses, this model investigates the pullout load due to mechanical deformation as well as additional friction caused by bending. Model predictions are compared with the experimental results of hooked fiber pullout data and reasonably good correlation is observed.

Mahmud G.H et al. (2013) Ultra High Performance Steel Fiber Reinforced Concrete (UHPFRC) is a relatively new construction material with high strength, fracture toughness and ductility. Although many aspects of UHPFRC have been investigated extensively, the size effects on structural strength of UHPFRC members remain largely unknown. This is mainly due to the lack of sufficient and reliable experimental data. This study investigates the size effects on flexural strength of similar notched UHPFRC beams under three-point bending tests. Nonlinear finite element simulations using the concrete damage plasticity (CDP) model in ABAQUS were also conducted, using material properties extracted from uniaxial tensile and compressive laboratory tests. It was found that the size effect on the beam nominal strength is little due to high ductility of UHPFRC. The numerical simulations using the CDP model can predict load–displacement curves and crack propagation process with good agreement with experimental data.

Park S.H. et al. (2014) reported the interfacial bond strength of long high-strength steel fibers embedded in ultra-high-performance concrete (UHPC) reinforced with short steel micro fibers was investigated by conducting single-fiber pullout tests. In particular, the influence of the addition of a shrinkage-reducing to a UHPC matrix on the pullout resistance of high-strength steel fibers was investigated. The addition of a shrinkage-reducing agent produced a noticeable reduction in the fiber pullout resistance owing to the lower matrix shrinkage, although the reduction of pullout resistance differed according to the type of fiber. Long smooth and twisted steel fibers were highly sensitive to the addition of the shrinkage-reducing agent whereas hooked fibers were not. Among the various high-strength steel fibers tested, twisted steel micro fibers showed the highest interfacial bond resistance, although twisted fibers embedded in UHPC showed slip softening pullout behavior rather than the typical slip hardening behavior observed in mortar.

2.3.1 Compressive Strength

In recent years, with the development of new plasticizing concrete admixtures and fine pozzolanic materials, it has become possible to produce high performance concrete (HPC) or even Ultra-high Performance Concrete (UHPC). For the production of UHPC, the pozzolanic materials (silica fume, ground granulated blast-furnace slag, fly ash) are widely utilized. Due to its high purity and high specific surface area, the pozzolanic reaction of silica fume is fast and could more effectively promote the strength development of concrete, compared to the other pozzolanic materials. Nevertheless, the new developments of nano technology guarantee that various forms of nano sized amorphous silica can be produced, which higher specific surface areas and activities have compared to conventional silica fume. Hence, a considerable investigation effort has been paid on clarifying their effect on the properties of concrete.

Graybeal (2008) reported the compressive strengths of nearly 1,000 specimens subjected to the following four different curing conditions:

- Steam curing at 194 °F (90 °C) and 95-percent relative humidity for 48 hours starting about 24 hours after casting.
- Steam curing at 140 °F (60 °C) for 48 hours starting about 24 hours after casting.
- Steam curing at 194 °F (90 °C) for 48 hours starting about 15 days after casting.
- Curing under laboratory conditions (73 °F (23 °C) and ambient humidity).

Most tests were conducted on 3- by 6-inch (76- by 152-mm) cylinders with the ends ground so that they were parallel within 1 degree. Tests generally used the procedures of ASTM C39, except the loading rate was increased to 150 psi/second (1 MPa/s), and a 6.5-inch (165-mm) - diameter spherical bearing plate was used.

The average measured compressive strengths at 28 days for six cylinders cured using methods a, b, c, and d were 28.0, 24.8, 24.8, and 18.3 ksi (193, 171, 171, and 126 MPa), respectively. Density of the UHPC ranged from 150 to 156 lb/ft^3 (2,400 to 2,500 kg/m^3). Within each curing regime, there was a slight increase in compressive strength as the density increased.

Graybeal also investigated the effect of cylinder and cube size on the measured compressive strength using 2 by 4 inch, 3 by 6 inch, 4 by 8 inch, and 3 by 6.5 inch (51 by 102 mm, 76 by 152 mm, 103 by 203 mm, and 76 by 165 mm) cylinders and 2 and 3.94 inch (51 and 100 mm) cubes. The measured strengths were all within 8 percent of the control 3 by 6 inch (76 by 152 mm) cylinder strength. The cubes had compressive strengths about 5 percent higher than the cylinders.

Similar results were also observed by **Orgass and Klug (2004)** the smaller cylinders and cubes had a larger standard deviation. 3.9 inch (100 mm) cubes had a 20 percent lower measured compressive strength than 2.0 inch (50 mm) cubes.

Skazlic et al. (2008) investigated the effect of cylinder size on the compressive strength of 10 different UHPC mixtures. Cylinder diameters were 2.75, 4, and 6 inches (70, 100, and 150 mm) with a length-to-diameter ratio of 2:1. Assuming a 4 by 8 inch (100 by 200 mm) cylinder as a standard, the authors proposed conversion factors of 1.05 to 1.15 for strengths measured on 2.75 by 5.5 inch (70 by 140 mm) cylinders and 0.85 to 0.95 for strengths measured on 6 by 12 inch (150 by 300 mm) cylinders.

ShihJY.et al. (2006) found that with the addition of nano silica in cement or concrete, even at small dosages, nano silica can significantly improve the mechanical properties of cementitious materials.

Nazari and Riahi (2011) showed that a 70% compressive strength improvement of concrete can be achieved with an addition of 4% (by mass of cement) of nano silica.

Li et al. (2004) found that when 3% and 5% nano silica were added to plain cement mortar, the compressive strength increased by 13.8% and 17.5% at 28 days, respectively.

Kazemi and Lubell (2012) also investigated compressive strength as a function of time after casting.The response of a locally sourced UHPC from central Canada was found to correspond to the relationship depending on the fiber content.

Schmidt and Fröhlich (2010) reported that irregularities in the loaded surface of specimens tested in compression caused a more pronounced decrease in the measured compressive strength in UHPC than was evident with conventional concrete.

Pimienta,P.et. al. (2009) observed the tests of UHPC in axial compression at elevated temperatures showed that the measured compressive strength decreases as the concrete temperature at testing increases. However, some or all of the strength is recovered after the specimens cool down.

Richard (1996) reported that compressive strengths as high as 80 ksi (550 MPa) can be achieved at atmospheric pressure and heat treating at 480°F (250°C). With pressure, compressive strengths as high as 117 ksi (810 MPa) are possible. With conventional production capabilities and curing at 194°F (90°C), strengths of 40 ksi (280 MPa) can be achieved.

The compressive strengths of UHPC at 3, 7 and 28 days versus the nano silica amounts are shown in **Fig. 2.1**. With the addition of nano-silica, the parabolic growth tendency of the compressive strength of UHPC can be observed.

Based on these optimal nano-silica amounts, the computed maximum strengths are 56.99 MPa, 69.63 MPa and 88.93 MPa for the compressive strength. Moreover, it is important to notice that computed maximum compressive strength at 28 days is 88.93 MPa, which is even smaller than the one with 4% of nano-silica (91.29 MPa). This should be attributed to the deviation of the regression equation. As can be noticed that the coefficient of determination of the one representing the compressive strength at 28 days is only 0.862. Hence, to accurately obtain the optimal nano-silica amount in this study, only 3.72%, 3.46%, 3.70%, 3.92% and 3.90% are selected to calculate the average value (3.74%), which represents the optimal amount for this nano silica in UHPC to theoretically get the best mechanical properties.

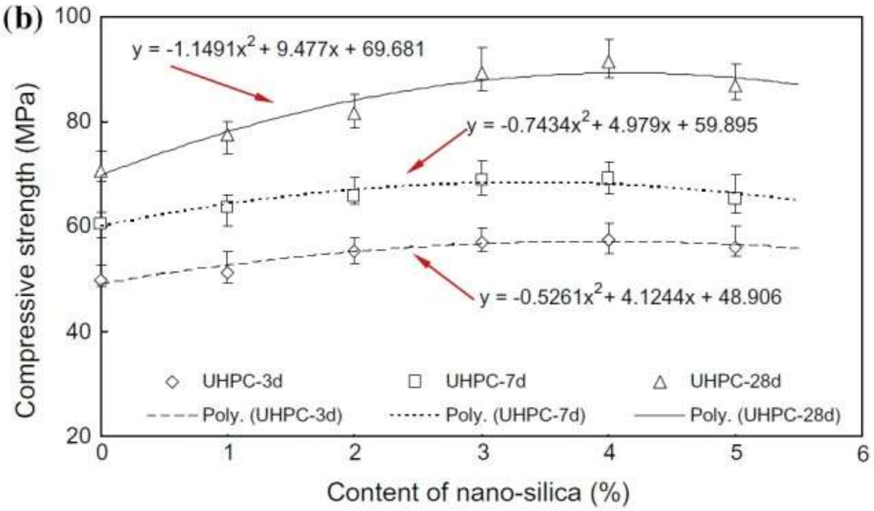


Fig.2.1: Variation of the compressive strength of UHPC after curing for 3, 7 and 28 days as a function of nano-silica amount

The compressive strengths improvement ratios of UHPC mixtures versus the nano-silica amount are illustrated in **Fig. 2.2.** It can be seen that, similarly to the flexural and compressive strengths results, the compressive strength improvement firstly increase with the increase of the nano-silica amount, and then decrease. Moreover, the effect of the nano-silica in improving the compressive strength of UHPC is more efficient at 3 days, comparing to that at 7 and 28 days. This phenomenon also implies that nano-silica can significantly promote the early hydration process of cement in UHPC.

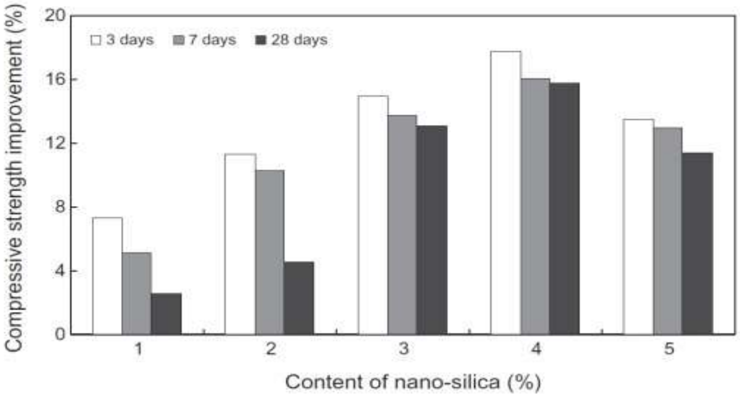


Fig. 2.2: Compressive strength improvement of UHPC with different nano-silica amount at 3, 7 and 28 days

2.3.2 Tensile Strength

In conventional structural design for concrete bridges, the tensile strength of concrete is assumed to be zero in reinforced concrete design and often taken as $6\sqrt{fc}$ in pre stressed concrete girder design.

The tensile strength of UHPC is higher than that of conventional concrete, and UHPC can exhibit sustained tensile strength after first cracking. The results of tests for tensile strength of UHPC, therefore, often report a value of first cracking strength as well as a peak post-cracking strength. Consequently, tensile strength takes on increasing importance as a property to consider in design.

According to **Graybeal, B., et. al. (2012)** tensile stress-strain response obtained from a readily available UHPC containing 2 percent by volume steel fiber reinforcement was captured by Graybeal and is shown in **figure 2.3**. The results shown were developed as part of a study.

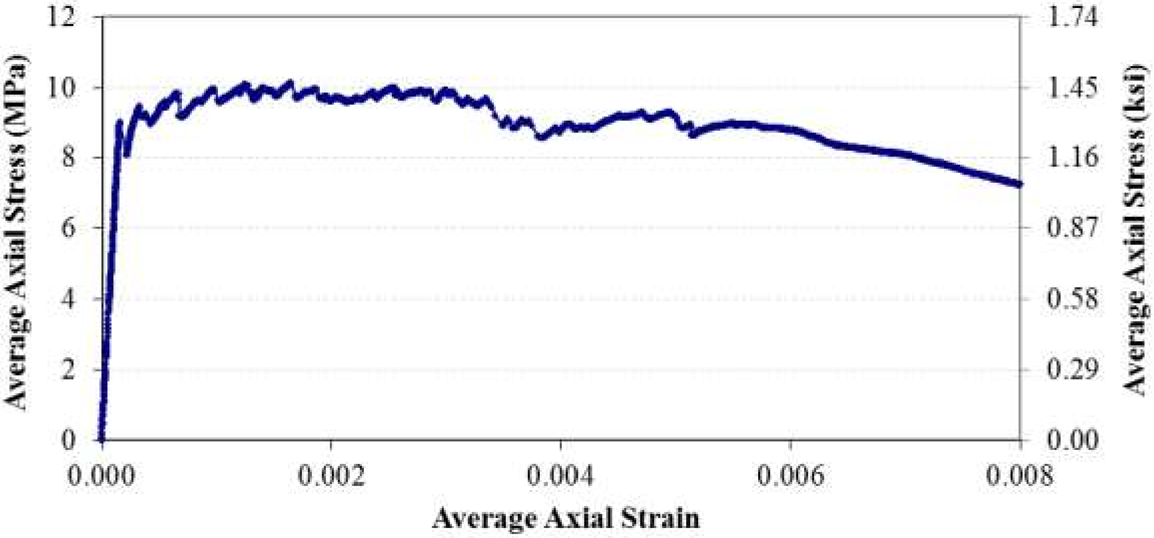


Figure 2.3: Graph Tensile stress-strain response of UHPC

Leutbecher, et. al. (2004) has proposed the idealized tensile stress-strain response shown in **Figure 2.4.** This response is based on direct tension tests of two UHPCs with multiple fiber contents. It is proposed as a conceptual illustration of the pre cracking and post cracking tensile stress-strain response of strain-hardening fiber reinforced concretes, such as UHPC. The behavior is divided into four phases. Phase

I is elastic behavior. Phase II is the phase wherein multiple tightly spaced cracks form in the UHPC matrix. The cracks occur individually as the stress in the matrix exceeds the matrix cracking strength. Phase III begins at the strain level where additional cracking between existing cracks is unlikely. Individual cracks widen in this phase. Lastly, Phase IV begins when an individual crack has reached its strain limit and the fibers bridging that crack begin to pull out of the matrix. In a strain-hardening fiber-reinforced concrete, the fiber bridging strength where localization occurs is greater than the cracking strength where multi cracking occurs.

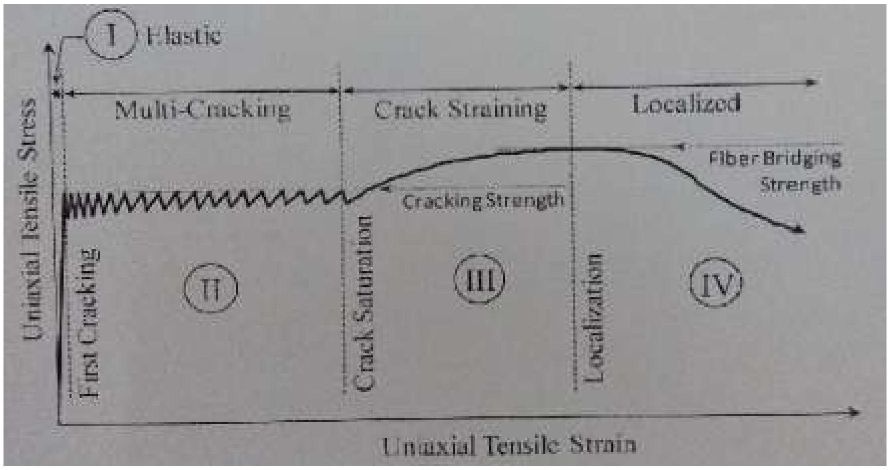


Fig. 2.4: Graph Idealized uniaxial tensile mechanical response of a UHPC

Qian, S.et al (2008) Standard tensile test methods designed to assess the cracking strength of conventional concrete may be appropriate for assessing the first cracking strength of UHPC, but are unlikely to be appropriate for quantitatively assessing the post-cracking tensile response of UHPC. The ASTM C78 Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading) and ASTM C496 Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens fall into this category. Both test methods include assumptions of mechanical behaviors that are not consistent with strain-hardening fiber reinforced concretes and thus are likely to overestimate the tensile strength of the UHPC.

Wille, K.et al (2012) has proposed a modified version of ASTM C496. The modified test method includes a requirement to monitor the first cracking of the UHPC during the test and calculate the splitting tensile strength based on the observed first cracking load.

Flexure-based test methods have been proposed, and in some cases, standardized. ASTM C1018, ASTM C1609, and RILEM TC 162-TDF all present test methods for use in determining the tensile response of fiber-reinforced concretes. Methods have been proposed for analyzing the test results so as to develop uniaxial tensile response curves. However, these types of flexure tests have been demonstrated to be susceptible to over indications of strength as a result of the use of inappropriate support conditions.

A variety of direct tension test methods have been developed. In a direct tension test, the UHPC specimen is loaded in uniaxial tension and thus the tensile response can be directly captured by measuring the load on and the strain experienced by the specimen. Direct tension tests can be divided into two groups, namely, tests that allow rotation of the ends of the test specimen and tests that do not. The tests with rotation might provide an indication of first cracking strength, but are not appropriate for assessment of post-cracking behaviors. This is because local inconsistencies in stiffness at the plane of the first crack result in rotation and fiber pullout at this Localized Uniaxial Tensile Stress Uniaxial Tensile Strain Localization Crack prior to the generation of a full set of additional cracks. Fixed-end tests that do not allow rotation at cracks are appropriate for capturing the full tensile stress-strain response. However, these tests are difficult to complete because of the bending stresses that can be imparted to the specimen during initial setup.

2.3.3 Flexural Strength

The calculated flexural resistance of concrete components is generally based on the conditions of equilibrium of forces and strain compatibility. The usable compressive strain in unconfined concrete is limited to a maximum value of 0.003. The shape of the stress-strain curve may be any shape that results in a prediction of strength in substantial agreement with test results. For simplification, a rectangular stress block for the compression zone is usually assumed. The tensile strength of the concrete is neglected. The applicability of this approach for use with UHPC has been addressed in several articles.

Graybeal et al (2006) tested a 36 inch (0.91 m) deep AASHTO Type II girder made of UHPC in flexure using four-point bending on a span length of 78.5 ft (23.9 m).The girder contained twenty four 0.5 inch (12.7 mm) diameter strands. Prior to reaching peak load, the girder achieved a deflection of almost 19 inches (480 mm) and failed by a combination of tensile fracture of the strands and pullout of the fibers.

Meade and Graybeal (2010) reported the results of sixteen 6 inch (152 mm) wide, 15-inch (381-mm) deep rectangular UHPC beams tested in four-point bending over a span length of 16 ft (4.88 m).The test variables were fiber content (0, 1, and 2 percent by volume) and quantity of conventional non pre stressed reinforcement (0.00 to 1.00 percent by area). Measured compressive strengths of the UHPC ranged from 24.7 to 29.4 ksi (170 to 203 MPa).

Beams containing 1 and 2 percent fiber reinforcement had higher first cracking strengths, better post-cracking flexural response, and higher peak loads than beams without fibers. Increasing the fiber content from 1 to 2 percent resulted in stiffer post-cracking response and higher peak loads. The beams containing no fibers failed when flexure-shear cracks extended into the compression region under the load points, leading to a shear failure of the flexural compression block in the shear span. The beams containing the fibers failed when the fibers pulled out across a critical crack and the reinforcing bars ruptured. No concrete crushing was noted.

Visage et al. (2012) reported the results of ten 6 inch (152 mm) square beams tested in flexure. Test variables included compressive strength, amount of flexural reinforcement, volume of steel fibers, and beam length. Test results were compared with traditional methods of estimating moment-curvature relationships.

Adeline and Behloul (1996) reported flexural tests of two 49.2 ft (15 m) long UHPC beams containing only flexural reinforcement. The beams contained eight or four 0.6 inch (15.2 mm) diameter strands. The beam with eight strands failed by crushing of the UHPC, whereas the beam with two strands failed by strand rupture. Both beams exhibited large deflections before failure. The authors used a nonlinear multilayer program to predict the moment deflection curves. They obtained very good agreement between the measured and calculated curves in both the elastic and plastic parts of the curves.

Maguire et al. (2009) tested two full size double-tee beams in flexure. The beams contained 0.7 inch (17.8 mm) diameter strands and UHPC without steel fibers. The measured strengths exceeded the calculated strengths using measured properties and a strength design approach. The authors concluded that the flexural design procedures of the AASHTO LRFD Specifications for I-girders are applicable to UHPC girders.

For strength design, **Stürwald and Fehling (2012)** developed a simplified approach. They used a triangular stress block in compression and a rectangular stress block for tension in the UHPC. This approach gave calculated strengths within 5 percent of the measured strength of three beams.

The stress–strain curves of UHPHFRC during the flexural test at7 and 28 days are shown in **Fig. 2.5 (a) and (b).** Similarly to the results shown in the literature, the addition of steel fibers (2 vol. %) can not only enhance the ultimate flexural strength, but also improve the energy absorption capacity of the designed UHPHFRC.

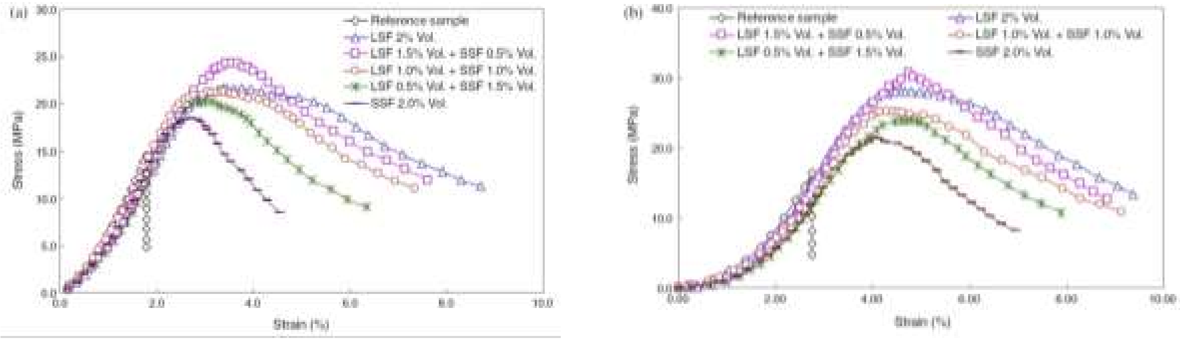


Fig. 2.5: Stress–strain curve of UHPHFRC under flexural test: (a) after curing for 7 days; and (b) after curing for 28 days

This should be attributed to the fact that the additional steel fibers can bridge cracks and retard their propagation, which could change the fracture behavior of concrete from brittle to plastic and significantly increase the ultimate flexural strength of concrete .Moreover, it is important to notice that the flexural properties of the specimen strongly depend on the fractions of the long and short steel fibers in the total fiber amount. As can be seen in **Fig. 2.5 (a) & (b),** the ultimate flexural strengths of the concrete with long steel fibers (1.5 vol. %) and short steel fibers (0.5 vol. %) at 7 and 28 days are always the largest, which are 24.3 MPa and 30.9 MPa, respectively. When only short steel fibers are utilized (2 vol. %), the ultimate flexural strengths at 7 and 28 days reduce to around18.4 MPa and 21.5 MPa, respectively. This can be explained by the following **two reasons**:

(1) Short fibers can bridge micro-cracks more efficiently, because they are very thin and their number in concrete is much higher than that of the long steel fibers, for the same fiber volume. Hence, when the micro-cracks are just generated in the concrete specimen, the short steel fibers can effectively bridge the micro-cracks. As the micro-cracks grow and merge into larger macro-cracks, the long steel fibers become more and more active in crack bridging. In this way, primarily the ductility can be improved, and partly also the flexural strength. Long fibers can therefore provide a stable post-peak response. Short fibers will then become less and less active, because they are being more and more pulled out, as the crack width increases.

(2) Long fibers are always well oriented between the two imaginary borders. If casting of concrete in layers is applied (these borders may also be the walls of the moulds). With such positions, the long fibers form a kind of a barrier for short fibers, and limit their space for rotation. The short fibers will therefore be somewhat better oriented when combined together with long fibers, than on their own see **Fig. 2.6.** Hence, more fibers distribute in the direction parallel to the force direction in the flexural test, and the mechanical properties can be significantly improved. Additionally, for the Sample with the largest ultimate flexural strength.

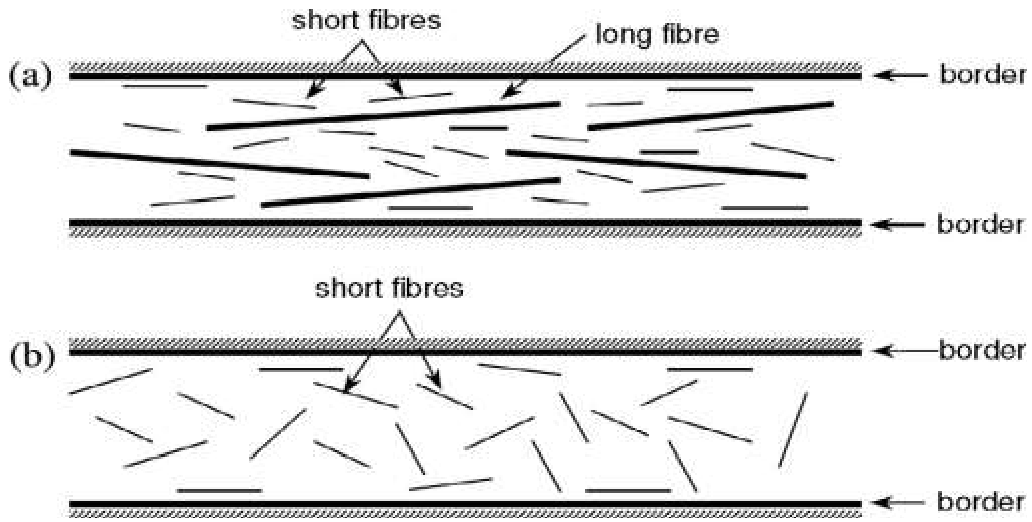


Fig.2.6: Orientation of short fibers between the walls of the mould (or any other borders in general) in combination with long fibers and (b) when they are alone on their own

It is important to notice that the stress quickly drops after reaching the stress peak. This should be attributed to the fact that the short steel fibers are less effective in bridging the macro-cracks and cannot provide a stable post-peak response. After calculating the area under the curves shown in **Fig. 2.5**, it is demonstrated that the energy absorption capacity of the batch with largest flexural strength may not be the largest, compared to the other mixtures in this research (161.4 units for reference sample and 139.8 units for LSF 1.5% Vol. + SSF 0.5% Vol. at 28 days).

In summary, the flexural properties of UHPHFRC largely depend on the proportions between the long and short steel fibers. In this study, although the sample with long steel fibers (1.5 vol. %) and short steel fibers (0.5 vol. %) shows the largest ultimate flexural strength, its post-peak response is not the highest observed.

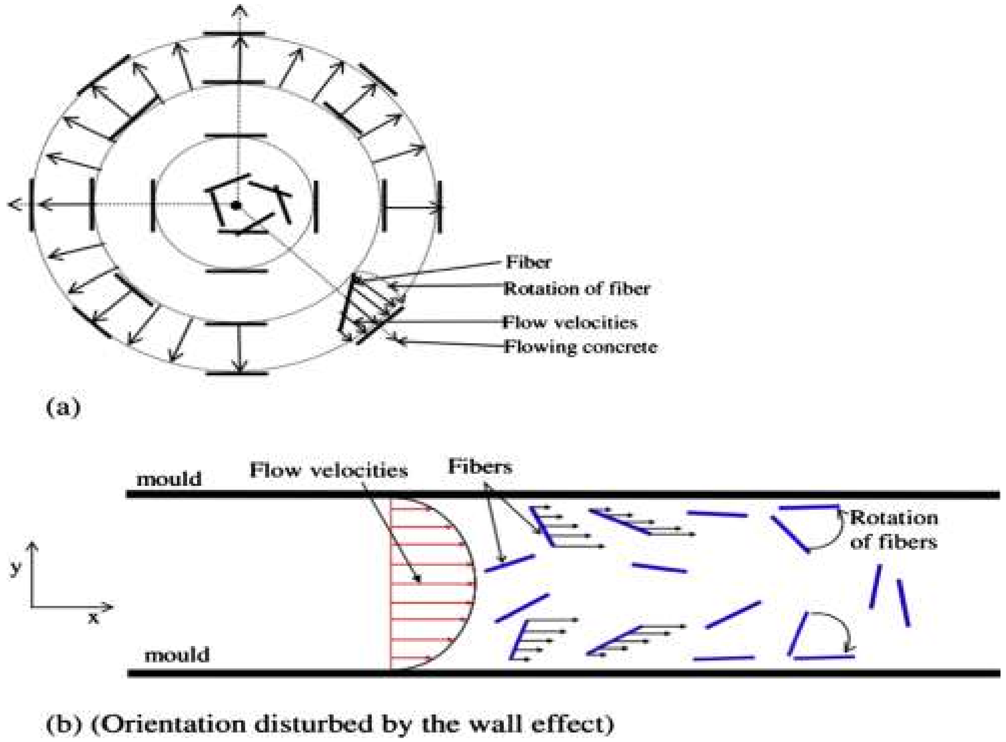


Fig. 2.7: Single type steel fibre orientations in (a) fountain flowing and (b) canal channel flowing

Based on the investigation by **Boulekbache B et al. (2010)** the mutual effects between hybrid fibres are utilized to produce the concrete,should be considered. It is known that different flow velocities affect the fibres and may cause them to rotate in such a way that the fibres reorient perpendicularly to the flow direction as shown in **Fig. 2.7.** In the presence of relative velocities in translation and rotation, the fluid exerts forces and momentums on the fibres. Hence, for the fresh concrete with single type fibres, the entire fibre orientation tends to be perpendicular to the flow direction in the fountain flowing mode, which can generate the largest resistant force and reduce the slump flow of the fresh concrete. Nevertheless, when hybrid fibres are added into the fresh concrete, the fountain flowing mode may be relatively disturbed. When casting concrete in moulds, the fibres close to the walls of the moulds tend to be parallel to the borders, which is named the ‘‘wall-effect’’.

Markovic I et al. (2006) shows in his study, when hybrid fibers are added into the fresh concrete, the long steel fibers can be treated as ‘‘imaginary borders’’ to the short steel fibers, and can relatively resist the rotation of the short fibers and reduce the resistance force in the fountain flow. Furthermore, the short steel fibers can also conversely restrict the rotation of the long steel fibers and further improve the ‘‘wall-effect’’ of long steel fibers. Therefore, the flowability of the hybrid fibers reinforced concrete is larger than that with only one type of steel fibers.

2.4 SLUMP FLOW

According to **Senff L, et al. (2014) the** slump flow of fresh UHPC mixes versus the amount of nano-silica is depicted in **Fig. 2.8.** The data illustrates the direct relation between the nano-silica amount and the workability of fresh UHPC. It is important to notice that with the addition of nano-silica, the slump flow of fresh UHPC decreases linearly. For the UHPC developed here, the slump flow value of the reference sample is33.75 cm, which sharply drops to about 22.5 cm when about 5% of nano silica is added. This behavior is in accordance with the results shown in, which indicates that the addition of nano silica greatly increases the water demand of cementitious mixes. One hypothesis explaining this is that the presence of nano-silica decreases the amount of lubricating water available within the inter particle voids, which causes an increase of the yield stress and plastic viscosity of concrete. Hence, in this study, the plastic viscosity of UHPC significantly increases with an increase of the nano-silica amount, which in turn causes that its workability obviously decreases.

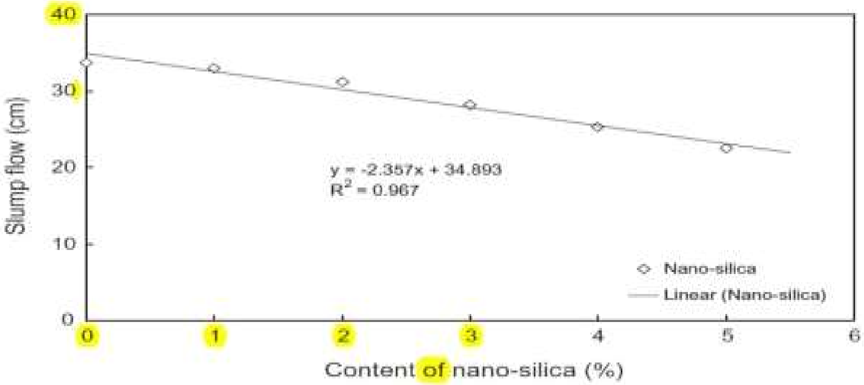


Fig.2.8: Slump flow of UHPC at different nano-silica additions

2.5 MIXING AND PLACING

2.5.1 Mixing

Graybeal (2011) has summarized the mixing of UHPC as nearly any conventional concrete mixer will mix UHPC. However, it must be recognized that UHPC requires increased energy input compared to conventional concrete, so mixing time will be increased. This increased energy input, in combination with the reduced or eliminated coarse aggregate and low water content, necessitates the use of modified procedures to ensure that the UHPC does not overheat during mixing. This concern can be addressed through the use of a high-energy mixer or by lowering the temperatures of the constituents and partially or fully replacing the mix water with ice. These procedures have allowed UHPC to be mixed in conventional pan and drum mixers, including ready mix trucks.

Mazanee et al. (2010) reported that mixing times for UHPC range from 7 to 18 minutes, which are much longer than those of conventional concretes. This impedes continuous production processes and reduces the capacity of concrete plants. Mixing time can be reduced by optimizing the particle size distribution, replacing cement and quartz flower by silica fume, matching the type of HRWR and cement, and increasing the speed of the mixer. The mixing time can also be reduced by dividing the mixing process into two stages. High-speed mixing for 40 seconds is followed by low-speed mixing for 70 seconds, for a total time of about 2 minutes.

Stiel et al. (2004) reported significant differences between horizontally and vertically cast beams when tested in three-point bending. The fibers in the vertically cast beams were aligned in layers normal to the casting direction. As a result, the splitting and flexural strengths were only 24 and 34 percent of the corresponding values for the horizontally cast beams. However, in a 39 inch (1 m) thick slab, the fibers were arranged randomly. The orientation of the fibers did not have a significant effect on the compressive strength and modulus of elasticity.

2.5.2 Placing

According to **Kim et al. (2008)** the method of placing UHPC has an influence on the orientation and dispersion of the fibers. The orientation did not affect the first cracking load but had an effect of up to 50 percent on the ultimate tensile strength in bending. The highest strengths were achieved when placement was made in the direction of the measured tensile strength.

Graybeal (2006) has summarized the placement of UHPC may immediately follow mixing or be delayed while additional mixes are completed. Although the dwell time prior to the initiation of the cement hydration reactions can be influenced by factors such as temperature and chemical accelerators, it frequently requires multiple hours before UHPC will begin to set. During extended dwell time, the UHPC should not be allowed to self-desiccate. Casting of fiber-reinforced concretes requires special considerations in terms of placement operations. UHPCs tend to exhibit rheological behaviors similar to conventional self-consolidating concretes, thus possibly necessitating additional form preparation but also allowing for reduced during-cast efforts. Internal vibration of UHPC is not recommended due to fiber reinforcement, but limited external form vibration can be engaged as a means to facilitate the release of entrapped air.


Source: New York State Department of Transportation

FIG. 2.9: Placing of UHPC

According **to Ozyildirim et al. (2011)** for the UHPC beams used on the Route 624 bridge over Cat Point Creek in Richmond, the contractor was required to use a plant that was pre qualified for UHPC production, and a representative from the UHPC producer was required to be present. The UHPC was mixed in 4 yd^3 (3 m^3) batches in an 8 yd^3 (6 m^3) twin shaft mixer and discharged into a ready-mixed concrete truck for delivery. About 20 to 25 minutes were required to load the mix, mix the UHPC, and discharge the mixer.

During discharge from the truck, cement balls were observed in the mix. This was attributed to exposure of the bags to moisture during storage. The mix was discharged into one end of the beam and allowed to flow. Only limited external vibration was applied for 1 or 2 seconds.

2.6 MECHANISM ANALYSIS OF HYDRATION AND MICRO STRUCTURE DEVELOPMENT

For normal cement grains, the initial hydration is largely limited to take place on the grain surface alone. With an ongoing hydration, the cement grain will be consumed and the hydration products that are deposited on the cement grain surface will grow thicker. With increasing curing time, due to the packed hydration products, the ionic transfer between the unhydrated cement particle and the surrounding solution is quite difficult, which limits cement hydration and generation of a dense C–S–H structure. In the present study, the hydration of the reference sample should be similar to such a process. Its porosity in hardened state is relatively high and the mechanical properties are comparatively low. When nano-silica is included into the hydration system of cement, an early pozzolanic reaction will take place on the silica surface to form C–S–H seeds as shown in **Fig.2.10 (a)**. Consequently, the formation of C–S–H phase is no longer limited to occur on the grain surface alone, and more C–S–H gel can be generated. In this study, due to the complexation of Ca_{2+} ions from the pore solution and polymer from the super plasticizer, the dormant period of cement hydration in UHPC can be significantly retarded. However, the addition of nano silica can efficiently compensate the retardation effect from super plasticizer. Based on the obtained results, it can be found that the nano silica amount is important for hydration and microstructure development of UHPC as shown in **Fig. 2.10 (b)**, when the amount of nano silica is low (e.g. less than 2%), the amount of the new generated

C–S–H seeds is low, which means that, to a large extent, most hydration still takes place on the surface of the cement particles. Hence, the porosity of the hardened cement matrix is still relatively large. When the amount of nano-silica is too high (e.g. 5% by the weight of binder), the sufficient amount of the generated C–S–H seeds can enable that the hydration products grow on their surfaces and result in a dense C–S–H gel. However, because the addition of nano-silica greatly increases the viscosity of cementitious mixes, a large amount of air can be entrapped into the cementitious system, which in turn increases the porosity of hardened concrete as shown **in Fig. 5.** When the nano-silica amount is optimal, the positive effect of the nucleation and the negative influence of the entrapped air can be well balanced. Therefore, there is an optimal nano-silica amount, at which the porosity of UHPC can be the lowest.

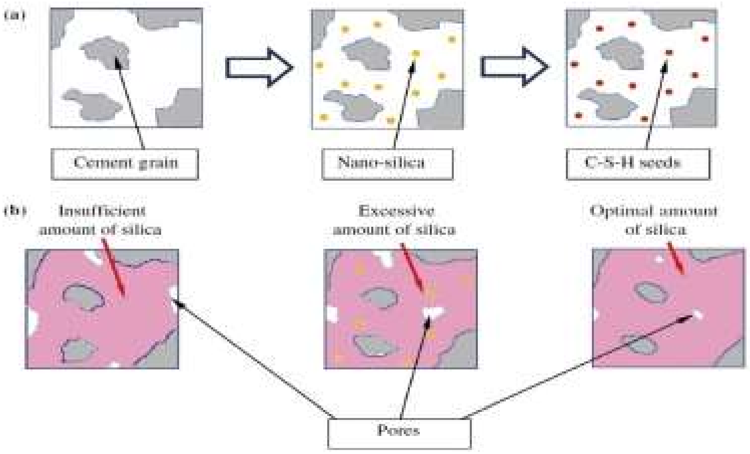


Fig.2.10: Schematic diagram of the nucleation effect of nano-silica on UHPC: (a) mechanism of the generation of C–S–H seeds and (b) effect of the nano-silica amount on the microstructure development and porosity of UHPC

2.7 CURING

Curing of UHPC considers two distinct components, specifically temperature and moisture. As with any cementitious composite material, maintaining an appropriate temperature is critical to achieving the desired rate for the cementitious reactions. In addition, given the low water content in UHPC, eliminating loss of internal water by sealing the system or maintaining a high humidity environment is also critical.

Graybeal (2011) reported the curing of UHPC occurs in two phases. Given that UHPC tends to exhibit a dormant period prior to initial setting, the initial curing phase consists of maintaining an appropriate temperature while precluding moisture loss until setting has occurred and rapid mechanical property growth is occurring. The second curing phase may or may not include elevated temperature conditions and a high moisture environment, depending on whether accelerated attainment of particular material characteristics is desired.

Graybeal (2011) also reported on an extensive program to determine material properties of UHPC using four different post-set curing procedures. These involved steam curing at 194 °F (90 °C) or 140 °F (60 °C) for 48 hours, starting about 24 hours after casting; steam curing at 194 °F (90 °C), starting after 15 days of standard curing and curing at standard laboratory temperatures until test age.

These three steam-curing methods increased the measured compressive strengths and modulus of elastic, decreased creep, virtually eliminated drying shrinkage, decreased chloride ion penetrability, and increased abrasion resistance. The enhancements achieved by the lower steam temperature and delayed steam curing were slightly less than achieved by steam curing at the higher temperature. The specimens steam cured at 194 °F (90 °C) after 24 hours reached their full compressive strengths within 4 days after casting.

More recent work by **Graybeal (2012)** has focused on characterizing the performance of ambient cured UHPC. This research stems from the recognition that accelerated curing in a steam environment is frequently not practical and also that the ambient-cured properties of UHPC are appropriate for many applications.

Ay et al (2004) compared the compressive strength of 4 inch (100 mm) cubes cured by the following three methods:

- Curing in water until 1 hour before testing.

- Curing in water for 5 days followed by air curing.
- Sealing the cubes in plastic sheeting and then storing them at 68 °F (20 °C) until tested.

The UHPC cubes stored in water followed by air curing had slightly higher compressive strengths than cubes cured by the other two methods.

Heinz and Ludwig et al (2004) showed that the compressive strength of UHPC can be increased considerably by using post-set heat curing. The heat curing at various temperatures between 149 and 356 °F (65 and 180 °C) produced 28-day compressive strengths as high as 41 ksi (280 MPa) compared with strengths of 25 and 27 ksi (178 and 189 MPa) when cured at 68 °F (20 °C). Higher curing temperatures resulted in higher compressive strengths. In addition, the strengths at the end of the curing period at about 48 hours after casting were about the same as the corresponding 28-day strengths. The authors also concluded that curing at 194 °F (90 °C) presented no danger of delayed ettringite formation.

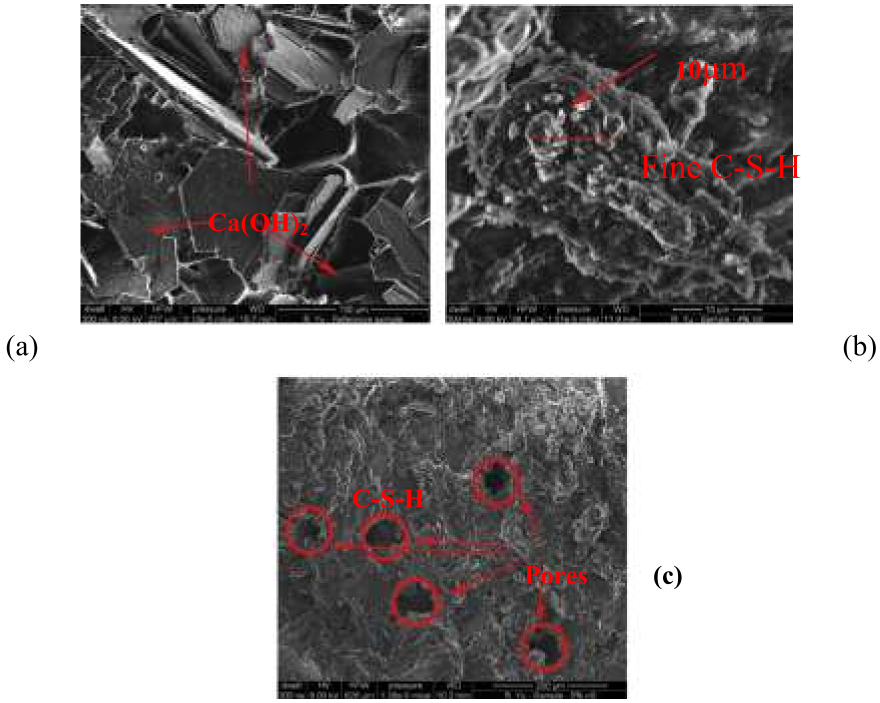
Schachinger et al. (2008) observed that initial curing at 68 °F (20 °C) for 5 days, followed by heat curing at 122 to 149 °F (50 to 65 °C), was the most favorable combination to achieve high strengths at ages up to 28 days. Compressive strengths in the range of 36 to 43.5 ksi (250 to 300 MPa) were achieved at ages of 6 to 8 years.

Heinz et al. (2012) achieved compressive strengths higher than 29 ksi (200 MPa) at an age of 24 hours after 8 hours storage at 68 °F (20 °C) followed by 8 hours at 194°F (90 °C) in water. Longer periods of initial storage or heat treatment resulted in higher strengths when ground-granulated blast-furnace slag was included in the UHPC. The authors obtained the highest strengths by including fly ash and autoclaving the UHPC for 8 hours at 300 °F (150 °C).

Massidda et al. (2001) showed that autoclaving at a temperature of 356 °F (180 °C) and 145 psi (1 MPa) with saturated steam produced higher compressive strength and flexural strengths compared with specimens cured at 68 °F (20 °C).

2.8 SEM ANALYSIS OF HARDENED UHPC

Scanning electron microscopy (SEM) is employed to study the morphology and microstructure of the reference sample and the samples with nano-silica additions. In **Fig. 2.11(a)** (reference sample), a large amount of well-developed Ca(OH)$_2$ plates can be observed, which implies that without nano-silica, the porosity of the hardened Concrete is relatively large and the Ca(OH)$_2$ has enough space to grow. In **Fig. 2.11(b)** (UHPC with 4% of nano-silica), one can observe a very dense structure in the hardened matrix with only a few air pores. The main hydration product of the cement matrix is the foil-like C–S–H gel, and no Ca(OH)$_2$ crystal can be easily found. Furthermore, fine C–S–H gel can be found, which is probably generated from the pozzolanic reaction of nano-silica with Ca(OH)$_2$. The microstructure of the specimen with 5% addition of nano-silica is presented in **Fig. 2.11(c)**, in which a number of pores can be observed. As explained previously, because the addition of nano-silica greatly increases the viscosity of cementitious mixtures, a large amount



(a) (b)

(c)

Fig.2.11: SEM images: (a) reference sample, (b) UHPC with 4% of nano-silica, and (c) UHPC with 5% of nano-silica.

of air can be entrapped in the cementitious system, which in turn increases the porosity of hardened concrete.

R.Yu et al. (2014) presents the effect of nano-silica on the hydration and microstructure development of Ultra-High Performance Concrete (UHPC) with a low binder amount. The design of UHPC is based on the modified Andreasen and Andersen particle packing model. The results reveal that by utilizing this packing model, a dense and homogeneous skeleton of UHPC can be obtained with a relatively low binder amount (about 440 kg/m^3). Moreover, due to the high amount of super plasticizer utilized to produce UHPC in this study, the dormant period of the cement hydration is extended. However, due to the nucleation effect of nano-silica, the retardation effect from super plasticizer can be significantly compensated. Additionally, with the addition of nano-silica, the viscosity of UHPC significantly increases, which causes that more air is entrapped in the fresh mixtures and the porosity of the hardened concrete correspondingly increases. In contrary, due to the nucleation effect of nano-silica, the hydration of cement can be promoted and more C–S–H gel can be generated. Hence, it can be concluded that there is an optimal nano-silica amount for the production of UHPC with the lowest porosity, at which the positive effect of the nucleation and the negative influence of the entrapped air can be well balanced.

The compressive strengths of the designed UHPHFRC with different hybrid fiber coefficients. As can be seen, compared to the reference sample, the additional steel fibers significantly increase the compressive strength of UHPHFRC. For instance, the compressive strengths of the reference sample (without steel fibers) at 7 and 28 days are 86.0 MPa and 99.0 MPa, respectively. When long steel fibers are added (2 vol. %) into the concrete, the compressive strength increase to about 114.3 MPa and 139.3 MPa after curing for 7 and 28 days, respectively. Moreover, similarly to the sample with long steel fibers (1.5 vol.%) and short steel fibers (0.5 vol.%) shows the largest compressive strengths, which are 117.1 MPa and 141.5 MPa after curing for 7 and 28 days, respectively. This should also be attributed to the combined effect of hybrid fibers in restricting the cracks development. Although the fibers are more efficient in improving the properties of concrete under tensile or flexural loads, they can also bridge the cracks that are generated in the compressive test and enhance the ultimate compressive strength of the concrete. However, it is important to notice that, when only short steel fibers are added into the concrete, the compressive strength slightly increases to 111.4 MPa and 120.8 MPa after 7 and 28 days, respectively. This should be attributed to the dimensions and aspect ratio of the short steel fibers. The length of the short steel fibers used in this study is 6 mm, which is less efficient in restricting the growth of macro cracks. Hence, the addition of short steel fibers (2 vol. %) is not as efficient as long steel fibers (2 vol. %) in improving the compressive strength of UHPHFRC.

The compressive strength results demonstrate that, based on the modified Andreasen & Andersen particle packing model, it is possible to produce UHPFRC with a relatively low cement or binder amount. It is clear that, with a lower binder amount, the compressive strength of the optimized UHPFRC is still comparable to the non-optimized UHPFRCs, which have a higher amount of binder.

The results shown by **Hassan et al. (2012)**, about 1200 kg/m^3 of binder are utilized to produce UHPFRC, and its compressive strength at 28 days is about 150 MPa. In this study, only about 620 kg/m^3 of binder is utilized in UHPHFRC, but its compressive strength can also reach around 142 MPa. Additionally, according to the cement efficiency of the designed UHPHFRC matrix can be calculated, which is about 0.17 MPa/kg/ m^3. Although this value is smaller than some of the eco concretes. It is much higher than that of other UHPCs. Hence, it is confirmed here that the optimized UHPHFRC matrix can have significantly reduced binder amount (without sacrificing the mechanical properties) and be more ‘‘green’’ in the terms of cement amount.

CHAPTER 3

EXPERIMENTAL PROGRAMME AND METHODOLGY

3.1 GENERAL

This chapter deals with the presentation of results obtained from various tests conducted on high performance fiber reinforced concrete. In order to achieve the objectives of present study, an experimental program was planned to investigate the effect of sea water and potable water on compressive strength, tensile strength, flexural strength, and permeability of UHPC.

3.2 MATERIALS

The properties of material used for making high performance fiber reinforced concrete mix are determined in laboratory as per relevant codes of practice. Different materials used in present study were Portland cement (43 grade as per IS code 8112-1989(re-affirmed in 2000)), silica fume, fine aggregates (size less than 2.36 mm), steel fiber and high range of super plasticizer. The aim of studying of various properties of material is used to check the appearance with codal requirements and to enable an engineer to design a cement mortar mix for an high strength. The description of various materials which were used in this study is given below:

3.2.1 Portland Cement

Ordinary Portland Cement (OPC) of J.K. make from a single lot was used throughout the course of the investigation. It was fresh and without any lumps. The physical properties of the cement as determined from various tests conforming to Indian Standard IS: 8112-1989 (Re-affirmed in 2000) are listed in **table 3.1**. Cement was carefully stored to prevent deterioration in its properties due to contact with the moisture. The various tests conducted on cement are initial and final setting time, specific gravity, fineness and compressive strength. The results of above said tests are shown in **table 3.1**.

Table 3.1: Properties of Ordinary Portland Cement (OPC)

Sr. No.	Characteristics	Values Obtained Experimentally	Value Specified By IS : 8112-1989 (re-affirmed in 2000)
1.	Standard Consistency, (percent)	27.5	-
2.	Fineness residue of Cement as retained on 90 Micron Sieve (percent)	2%	Not more than 10%
3.	Specific Gravity	3.08	-
4.	Soundness of cement (mm) by Le-Chatelier apparatus	2	Not more than10 mm
5.	Initial Setting Time (minutes)	120	Not less than 30
6.	Final Setting Time (minutes)	410	Not more than 600
7.	Compressive Strength (N/mm^2) 3 days 7 days 28 days	24.3 35.9 48.1	23 (minimum) 33 (minimum) 43 (minimum)

NOTE: It can be observed from above table that all the results satisfy the physical requirements of cement which is used in throughout the course of the investigation.

3.2.2 Silica Fume

Silica fume, also known as micro silica or condensed silica fume is a pozzolanic admixture. When used in concrete it will fill the void space between cement particles resulting in a more impermeable concrete. Silica fume is added as 25.71% of cement replacement by weight. Silica fume was provided by TGS Impex India Pvt Ltd and conformed the IS code 15388 (2003) specifications.

3.2.3 Fine Aggregates

According to size, the fine aggregate may be described as coarse, medium, and fine sands. The sand used for the experimental works is locally procured from Patiala and conformed to grading zone II (size less than 2.36 mm) .Depending upon the particle size distribution IS: 383-1970 has divided the fine aggregate into four grading zones (Grade I to IV). The grading zones become progressively finer from grading zone I to IV. It was coarse sand light grey in colour. Sieve analysis and physical properties of fine aggregate are tested as per IS: 383-1970 and results are shown in **table 3.2** and **table 3.3** respectively.

Table 3.2: Sieve Analysis of Fine Aggregate

Weight of sample taken =1000 gm.

Sr. No.	IS-Sieve size (mm)	Weight of aggregate retained (gm)	%age Retained	Cumulative % Retained	% Passing	Limits of zone II as per IS-383
1	4.75	0	0	0	100	90-100
2	2.36	0	0	0	100	75-100
3	1.18	196	19.6	19.6	80.4	55-90
4	600 µ	216	21.6	41.2	58.8	35-59
5	300 µ	436	43.6	84.8	15.2	8-30
6	150 µ	143	14.3	99.1	0.9	0-10
7	Pan	09	0.9	-	-	-
	Total	1000.00		ΣF =244.7		
			FM = 244.7 / 100 =2.45			

NOTE: Maximum size of Fine Aggregate is less than 2.36 mm is used in throughout the course of the investigation.

Table 3.3: Physical Properties of Fine Aggregate

Sr. No.	Characteristics	Value
1	Type	Natural Sand
2	Specific Gravity	2.62
3	Water Absorption	1.02 %
4	Moisture Content	0 %
5	Fineness Modulus	2.45
6	Grading Zone	II

3.2.4 Steel Fibers

Mild steel fibers having 30 mm length and 0.6 mm thickness i.e. aspect ratio (I/d) 50 which are corrugated and obtained through cutting of steel wires have been used. The fibers have been cut by fiber cutting machine to an accurate size. Properties of steel fiber used are tabulated in **table 3.4**.

Table 3.4: Properties of Steel Fibers

Average Thickness	0.6 mm
Length	30 mm
Density	7850 kg/m^3
Tensile Strength	8500 kg/ m^3
Shape	Crimped steel fiber

3.2.5 Super Plasticizer (High Range Of Water Reducers)

Super-plasticizers constitute a relatively new category and improved version of plasticizer. They are chemically different from normal plasticizers. Use of super-plasticizer permits the reduction of water to the extent up to 30 percent without reducing workability in contrast to possible reduction up to 15 percent in case of plasticizers. The mechanism of action of super-plasticizer is more or less same as in case of ordinary plasticizer. The super-plasticizer is more powerful as dispersing agents and they are high water reducers.

Master Glenium SKY 8777 **shown in Fig 3.1** make was used throughout the course of the investigation. It is a viscosity modifying agent in self-compacting concrete.

Master Glenium SKY 8777 ensures that rheoplastic concrete remains workable in excess of 90 minutes at +25°C. Workability loss is dependent on temperature, and on the type of cement, the nature of aggregates, the method of transport and initial workability.

Optimum dosage of Master Glenium SKY 8777 should be determined with trial mixes. As a guide, a dosage range of 500 ml to 1500 ml per 100 kg of cementitious material is normally recommended. In these experiment dosage of 11 ml per 0.875 kg of cementitious material have been used.

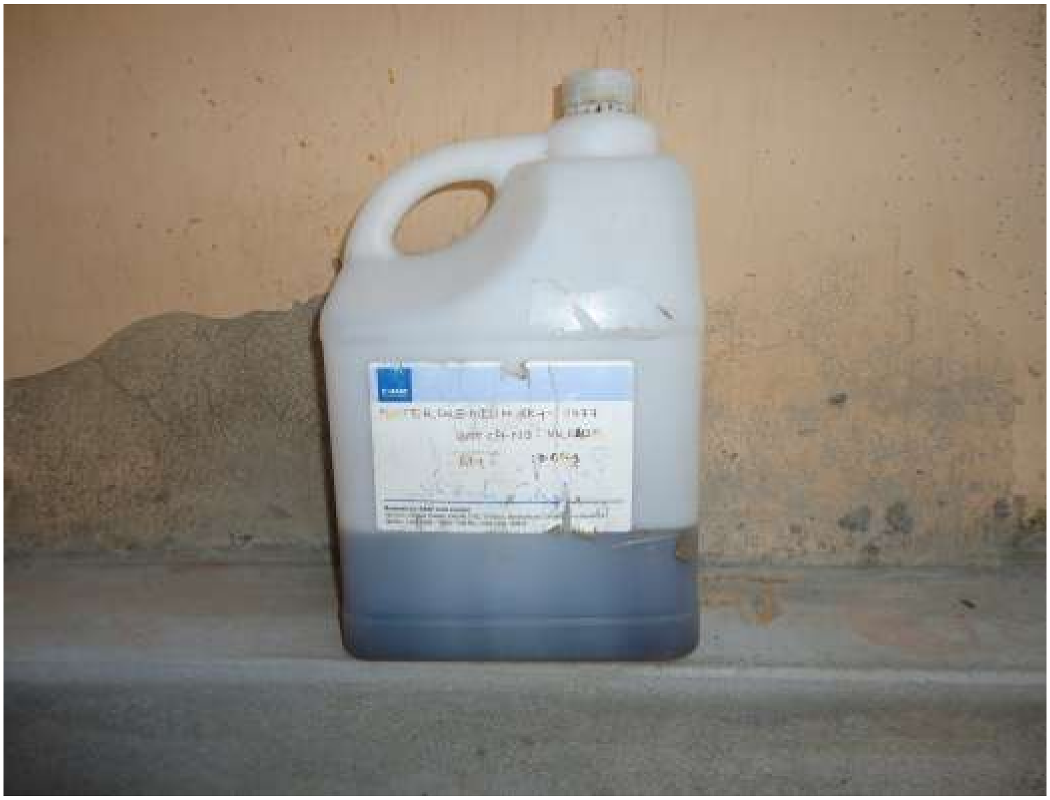


Fig. 3.1: Master Glenium SKY 8777

Effects of over dosage

A severe over-dosage of Super plasticizer can result in the following consequences:

- Extension of initial and final set.
- Bleed/segregation of mix, quick loss of workability.
- Increased plastic shrinkage.

The super-plasticizer Master Glenium SKY 8777 procured from BASF India Limited Construction Chemicals Division,Plot No.37, Chandivali Farm Road, Chandivali, Andheri (East) Mumbai – 400072 India was used in present study. The technical data provided by manufacturer is given in **table 3.5**.

Table 3.5: Properties of Super Plasticizer

Sr. No.	Characteristics	Value
1.	Appearance	Light brown liquid
2.	Chloride content	< 0.2%
3.	Relative Density	1.24 ± 0.02 at 25°C
4.	pH	≥ 6

3.2.6 Water

Water is an important and least expensive ingredient of concrete. A part of mixing water is utilized in the hydration of cement to form the binding matrix in which the inert aggregates are held in suspension until the matrix has hardened. If water is fit for drinking it is fit for making concrete. This does not appear to be a true statement for all conditions. Some waters containing a small amount of sugar would be suitable for drinking but not for mixing concrete and conversely water suitable for making concrete and not necessarily be fit for drinking. If water is not obtained from source that have proved satisfactory, the strength of concrete made with questionable water would be compared with similar concrete made with pure water. If the pH value of water is lies between 6 and 8 and water is free from organic matter is acceptable for mixing the concrete.

3.2.6 a) Potable Water

Potable water is generally considered satisfactory. In the present investigation, tap water is used for both mixing and curing purposes.

Table 3.6: Tolerable Concentrations of Some Impurities in Mixing Water

Sr.No.	Impurity	Tolerable Concentration
1	Sodium and potassium Carbonates and bi-carbonates	1000 ppm (total). If this is exceeded, it is advisable to make tests both for setting time and 28 days strength.
2	Chlorides	10000 ppm
3	Sulphuric anhydride	3000 ppm
4	Calcium chloride	2 percent by weight of cement in non-pre-stressed concrete.
5	Sodium iodate, sodium sulphate, sodium arsenate, sodium borate	Very low
6	Sodium sulphide	Even 100 ppm warrants testing.
7	Sodium hydroxide	0.5 per cent by weight of cement provided quick set is not induced.
8	Salt and suspended particles	2,000 ppm. Mixing water with a high content of suspended solids should be allowed to stand in a settling basin before use.
9	Total dissolved salts	15000 ppm.
10	Organic material	3000 ppm. Water containing humic acid are reported to have seriously impaired the strength of concrete 780 ppm of humic acid are reported to have seriously impaired the strength of concrete. In the case of such waters therefore, further testing is necessary.
11	pH	Shall not be less than 6.

3.2.6 b) Sea Water

Seawater or **salt water** is water from a sea or ocean. On average, seawater in the world's oceans has a salinity of about 3.5% (35 g/L), which is a specific gravity of about 1.025. This means that every kilogram (roughly one litre by volume) of seawater has approximately 35grams of dissolved salts (predominantly sodium (Na+) and chloride (C1-) ions). Seawater is denser than both fresh water and pure water (density 1.0 g/ml at 4 °C (39 °F)) because the dissolved salts increase the mass by a larger proportion than the volume. The freezing point of seawater decreases as salt concentration increases. At typical salinity, it freezes at about −2 °C (28 °F). Seawater pH is typically limited to a range between 7.5 and 8.4. However, there is no universally accepted reference pH-scale for seawater and the difference between measurements based on different reference scales may be up to 0.14 units. Sea water made in the lab by dissolving chemicals in distilled water as shown in **table 3.7**.

Table: 3.7: Compositions of Chemicals in preparations of artificial sea water

Sr. no.	Description of salt	Chemical formula of salt	Molecular weight	G/kg of solution
1	Sodium Chloride	NaCl	58.44	23.926
2	Sodium Sulphate	Na_2SO_4	142.04	4.008
3	Potassium Chloride	KCl	74.56	0.677
4	Sodium Bicarbonate	$NaHCO_3$	84.00	0.196
5	Potassium Bromide	KBr	119.01	0.098
6	Boric Acid	H_3BO_3	61.83	0.026
7	Sodium Fluoride	NaF	41.99	0.003

3.3 PREPARATIONS OF SAMPLE

The quantities of cement, silica fume, fine aggregate, fibers, HRWR and water for each batch was weighed separately. Firstly, cement and Silica fume were mixed separately to form uniform colour and then add the fine aggregate in dry state and mix in Digi Mortar Mixture for minimum 2 minutes at low speed to make a uniform colour. HRWR and water were mixed separately and added to the dry mix slowly through bur rete. Firstly, 50% to 70% of mix solution was added to the dry mix and then mixed thoroughly for 1 to 2 minutes at high speed. Then the remaining solutions added in mixture and again mixes thoroughly at high speed for 3-4 minutes and at last add fiber in mix and mixed again. After properly mixing the cement mortar was filled into the required moulds of cube size 70.6 X 70.6 X 70.6 mm (for compressive and tensile strength), moulds of beam Size 40 mm x 40 mm x 160 mm (for flexural strength) and cylindrical moulds of 100 mm (diameter) X 200 mm (long) (for Rapid Chloride Permeability Test and Water Sorptivity Test). No vibration is required as it is a self-compacted concrete. The surface of the mortar was finished it self and leveled with the top of the moulds. The finished specimens were left to harden in air for 24 hours. The specimens were removed from the moulds after 24 hours of casting and were placed in potable water and sea water at room temperature in the laboratory. The age of sample shall be calculated from the time of addition of water to the dry ingredients. At least three specimens shall be made for testing at each selected age.

3.4 TEST METHODS

To obtain the different parameters like compressive strength, tensile strength flexural strength and permeability, first of all select the material volume for high performance fiber reinforced concrete by hit and trail method in lab also take reference from different literature and fix the proportions according to trial no.5 by varying the proportion of cement and silica fume and fix the other ingredients. Trial No.5 gives best results in compressive strength after 7 days. Detail descriptions of various trial tests conducted in lab are given in **table 3.8:-**

Table 3.8: Mix Proportion of Different Ingredients of HPFRC

S. No.	Cement (in gm)	Silica Fume (in gm)	Sand (in gm)	Fiber (in gm)	HRWR (in gm)	Water (in gm)	7 days Compressive strength (in N/mm^2)
Trial 1	900	200	950	150	11	220	76.4
Trial 2	800	300	950	150	11	220	61.56
Trial 3	700	400	950	150	11	220	45.89
Trial 4	850	250	950	150	11	220	75.13
Trial 5	**875**	**225**	**950**	**150**	**11**	**220**	**91.4**

3.4.1 Compressive Strength

Determination of compressive strength of the high performance fiber reinforced concrete with mortar cubes of size 70.6 X 70.6 X 70.6 mm as shown in **Fig. 3.2** were casted for determination of compressive strength under different curing regime i.e. sea water and potable water up to age of testing. Details regarding number of specimen given in **table 3.9** below:

Table 3.9: Specimen for Potable Water and Sea Water Curing

Sr. No	No. of Specimens for potable water	No. of Specimens for Sea Water	Remarks
1	3	3	For 7 days testing
2	3	3	For 28 days testing

Total specimens = 12

At the test age of the specimens stored in water shall be tested immediately on removal from the water and while they are still in the wet condition. Surface water and grit shall be wiped off the specimen and any projecting fins removed which are to be in contact with the compression platens. In the case of cubes the specimens shall be placed in the machine in such a manner that the load shall be applied to the opposites sides of the cubes as cast that is not to the top and bottom. The axis of the specimen shall be carefully aligned with the centre of thrust of the spherically seated platen. No packing shall be used between the faces of the test specimen and the steel platen of the testing machine see **Fig.3.3**. The load shall be applied without shock and increased continuously at a rate of approximately 70 KN/min specified IS: 516 - 1959. Until the resistance of specimen increasing load break down and no greater load can be sustained. The maximum load applied to the specimen shall than be recorded and the appearance of the concrete and any unusable feature to the type of failure shall be noted.

Fig. 3.2: Cube Specimens of Size 70.6 mm x 70.6 mm x 70.6 mm

Fig. 3.3: Compressive Strength Testing of Cube under UTM

Thus the compressive strength was found by following formula:

$$f = \frac{F}{A}$$

Where: f = Compressive Strength (MPa),

F = Ultimate Load (N)

A = Cross-Sectional Area perpendicular to loading direction (mm^2)

3.4.2 Splitting Tensile Strength

Determination of tensile strength of the high performance fiber reinforced concrete with mortar cubes of size 70.6 X 70.6 X 70.6 mm were casted for determination of tensile strength under different curing regime i.e. sea water and potable water up to age of testing. Details regarding number of specimen given below in **table 3.10**:

Table 3.10: Specimen for Potable Water and Sea Water Curing

Sr. No	No. of Specimens for potable water	No. of Specimens for Sea Water	Remarks
1	3	3	For 7 days testing
2	3	3	For 28 days testing

Total specimens = 12

Fig. 3.4: Split Tensile Strength Testing of Cube under UTM

The splitting tensile test are well known indirect test used for determining the tensile strength of concrete **see Fig. 3.4**. This test performed on cubes by splitting in the following ways:

A) Along its middle parallel to the edges by applying two opposite compressive forces **see Fig. 3.5 (a)** through 15 mm square bars of sufficient length. Tensile strength is determined by using the formula 0.642 P/S^2.Where P is the load at failure and S is the side of the cube.

B) Along one of the diagonal planes by applying compressive forces along two opposite edges **see Fig. 3.5 (b).** Tensile strength is determined by using the formula 0.519 P/S^2.Where P is the load at failure and S is the side of the cube.

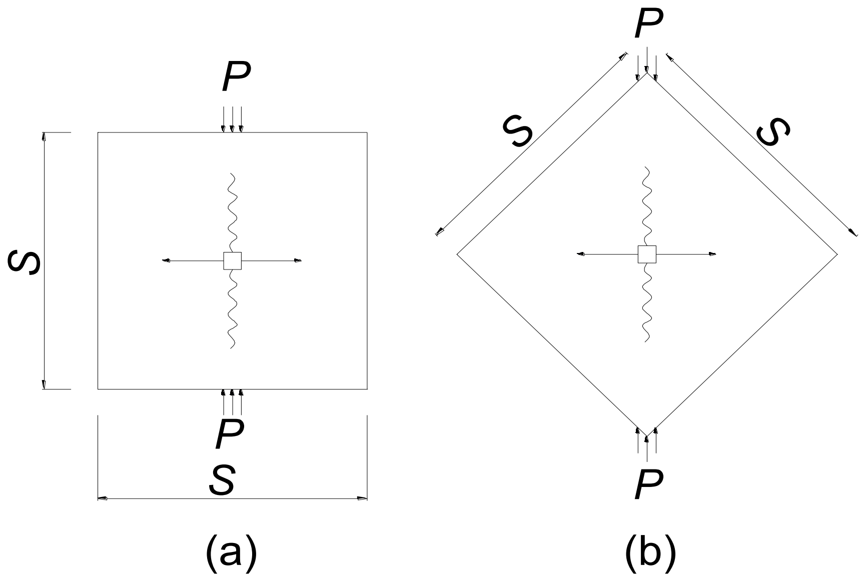


Fig. 3.5: Split Strength from Testing of Cube

3.4.3 Flexural Strength

Flexural Strength of material is resistance to deformation when it is subjected to lateral loading. The flexural test is more easily carried out than crushing test for use in field, since in this test much smaller loads are required. It is also known as

Modulus of rupture, bend strength, fracture strength. Flexural test intended to give flexural strength of mortar in tension. To determine the flexural strength of high performance fiber reinforced concrete, the specimens of size 40 X 40 X 160 mm **see Fig.3.6** were casted under different curing regime i.e. sea water and potable up to age at testing. Detail regarding number of specimen is given in **table 3.11**.

Table 3.11: Specimen for Potable Water and Sea Water Curing

Sr. No	No. of Specimens for potable water	No. of Specimens for Sea Water	Remarks
1	3	3	For 7 days testing
2	3	3	For 28 days testing

Total specimens = 12

Fig. 3.6: Mould Specimens of Size 40 mm x 40 mm x 160 mm

The specimens of size 40 mm x 40 mm x 160 mm were tested at the ages of 7 & 28 days. The position of specimen during flexural testing by centre point loading method was shown in **Fig. no. 3.7** and in schematic diagram shown in **Fig. no. 3.8.** The tests were performed on Universal Testing Machine (UTM). During testing loading is applied gradually at the rate of 2.65 KN/min. and the effective length of beam is taken as 120 mm without shock till the failure of the specimen occurs.

Fig. 3.7: Flexural Strength Testing of Specimen under UTM by Centre Point Loading

The flexural strength under centre point loading can be finding out by the formula given below:

$$\sigma = \frac{3FL}{2BD^2}$$

Where

σ = Stress at outer fiber (N/mm^2),

F = Load at fracture point (N),

L = Length of supported span (mm),

B = Width (mm),

D = Thickness (mm).

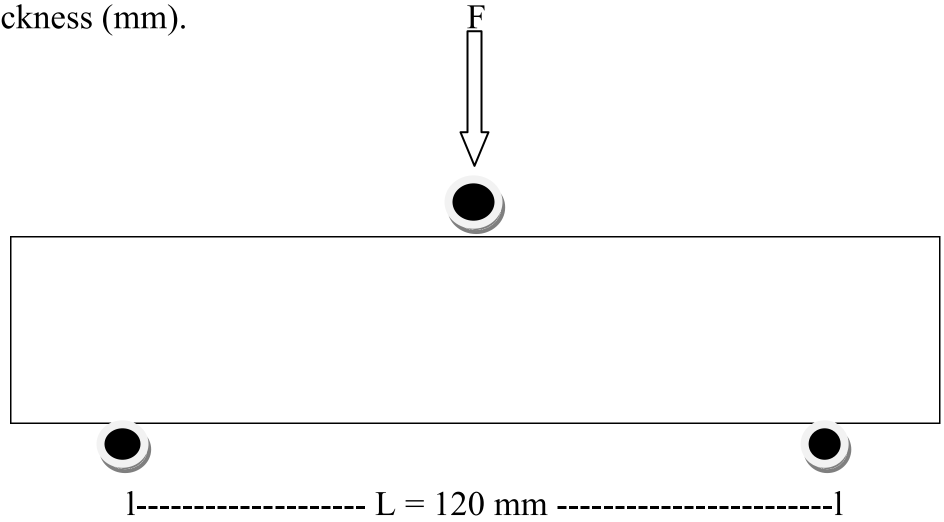


Fig.3.8: Schematic Diagram for Centre Point Loading method

3.4.4 Rapid Chloride Permeability Test

This test method consists of monitoring the amount of electrical current passed through 2-in. (51-mm) thick slices of 4-in. (102-mm) nominal diameter cores or cylinders during a 6-h period. A potential difference of 20 V dc is maintained across the ends of the specimen, one of which is immersed in a sodium chloride solution, the other in a sodium hydroxide solution. The total charge passed, in coulombs, has been found to be related to the resistance of the specimen to chloride ion penetration. Sample age has significant effects on the test results, depending on the type of concrete and the curing procedure. Most concretes, if properly cured, become progressively and significantly less permeable with time. Numerical results of this test (total charge passed, in coulombs) can be used as a basis for determining the accept-ability of a concrete mixture as shown in **table 3.12**. Factors such as the ingredient materials used and method and duration of curing of test specimens affect results of this test.

The test is not valid for specimens containing reinforcing steel positioned longitudinally that is, providing a continuous electrical path between the two ends of the specimen.

Table no. 3.12: Provide a qualitative relationship between the results of this test and the chloride ion penetrability of concrete

Sr. No.	Charge Passed (in coulombs)	Chloride ion Penetrability
1	>4000	High
2	2000-4000	Moderate
3	1000-2000	Low
4	100-1000	Very Low
5	<100	Negligible

To determine the permeability of high performance fiber reinforced concrete with cylindrical specimens of 100 mm (diameter) X 200 mm (long) were casted under different curing regime i.e. potable water and sea water to age at testing i.e.7 days and 28 days. Detail regarding number of specimen given below **in table 3.13**.

Table 3.13 Specimen for Potable Water and Sea Water Curing

Sr. No.	No. of Specimens for potable water	No. of Specimens for Sea Water	Remarks
1	3	3	For 7 days testing
2	3	3	For 28 days testing

Total specimens = 12

Fig. 3.9 (a): Cylindrical Mould for Specimens of Size 100 mm x 200 mm

Fig. 3.9 (b): Casted Cylindrical Specimens of Size 100 mm x 200 mm

For rapid chloride permeability test (RCPT) when the casted cylindrical specimens of 100 X 200 mm attained the age of testing i.e.7 days and 28 days, then specimen cutter machine as shown in **Fig. 3.10.**The cylindrical specimen of 51 mm (long) x 100 mm (diameter) size were cut from cores as shown in **Fig. 3.11**.

Fig. 3.10: Cylindrical Specimens Cutter Machine

Fig. 3.11: Cylindrical Specimens of Size 51 mm x 100 mm

The specimens at the age of testing i.e. 7 days and 28 days were placed in curing tank for at least 24 hours before the RCPT. Then specimens were placed in the vacuum desiccators' bowl as shown in **Fig 3.12** which illustrates the setup of the vacuum pump, desiccators with stopcock, vacuum gauge and valve and the de-aerated water container after the water has filled the desiccators. The vacuum was maintained in the desiccators bowl for 3 hours.

Fig 3.12: Vacuum Desiccators' Bowl

The specimens were removed from the desecrator, dried and placed in gasket. The specimen is then placed in the testing apparatus as shown in **Fig. 3.13** where one end of the specimen is exposed to a solution containing sodium chloride (NaCl) and the other end is exposed to a solution containing sodium hydroxide (NaOH) as shown in schematic **Fig. 3.14**. To increase the rate of chloride penetration into the polymer modified cement mortar specimens, thus speeding up the test, a constant 20 V potential were applied across the specimens. The current across the specimens were measured after the 6-hour test and then permeability was evaluated.

Fig.3.13: Rapid Chloride Permeability Test Setup

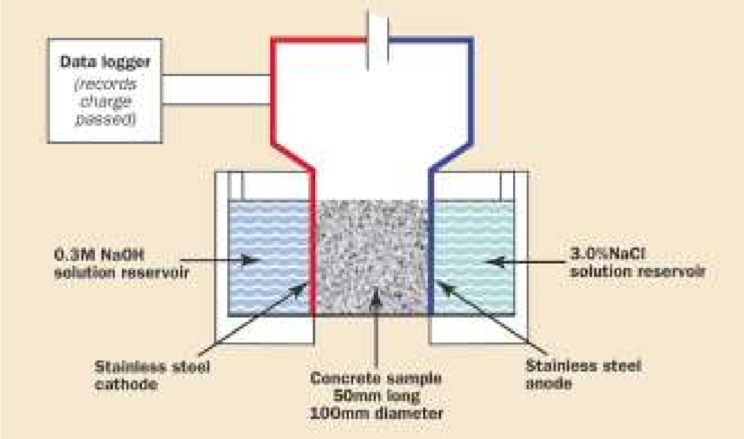


Fig.3.14: Schematic of Rapid Chloride Permeability Test Setup

3.5 Water Sorptivity Test

This test method is used to determine the rate of absorption (sorptivity) of water by hydraulic cement concrete by measuring the increase in the mass of a specimen resulting from absorption of water as a function of time when only one surface of the specimen is exposed to water. The specimen is conditioned in an environment at a standard relative humidity to induce a consistent moisture condition in the capillary pore system. The exposed surface of the specimen is immersed in water and water ingress of unsaturated concrete is dominated by capillary suction during initial contact with water.

The performance of concrete subjected to many aggressive environments is a function, to a large extent, of the penetrability of the pore system. In unsaturated concrete, the rate of ingress of water or other liquids is largely controlled by absorption due to capillary rise. This test method is based on that developed by Hall who called the phenomenon “water sorptivity.”

The water absorption of a concrete surface depends on many factors including:

a) Concrete mixture proportions.

b) The presence of chemical admixtures and supplementary cementitious materials.

c) The composition and physical characteristics of the cementitious component and of the aggregates.

d) The entrained air content;

e) The type and duration of curing.

f) The degree of hydration or age.

g) The presence of micro cracks.

h) The presence of surface treatments such as sealers or form oil.

i) Placement method including consolidation and finishing.

j) Water absorption is also strongly affected by the moisture condition of the concrete at the time of testing.

This method is intended to determine the susceptibility of an unsaturated concrete to the penetration of water. This test method is used to measure the water absorption rate of the concrete surface concrete.

In this test method, only one surface is exposed to water at room temperature while the other surfaces are sealed simulating water absorption in a member that is in contact with water on one side only. This test Method is used to estimate the maximum amount of water that can be absorbed by a dry specimen and therefore provides a measure of the total, water permeable pore space.

The surface to be exposed during testing shall not be marked or otherwise disturbed in such a manner as may modify the absorption rate of the specimen. The average test results on at least 2 specimens shall constitute the test result. The test surfaces shall be at the same distance from the original exposed surface of the concrete.

Remove the specimen from the storage container and record the mass of the specimen to the nearest 0.01 g before sealing of side surface. Measure at least four diameters of the specimen at the surface to be exposed to water. Measure the diameters to the nearest 0.1 mm and calculate the average diameter to the nearest 0.1 mm. Seal the side surface of each specimen with a suitable sealing material. Seal the end of the specimen that will not be exposed to water using a loosely attached plastic sheet. The plastic sheet can be secured using an elastic band or other equivalent system **see Fig. 3.15.**To determines water absorption as it is a function of time. Conduct the absorption procedure at 23 ± 2°C with tap water conditioned to the same temperature.

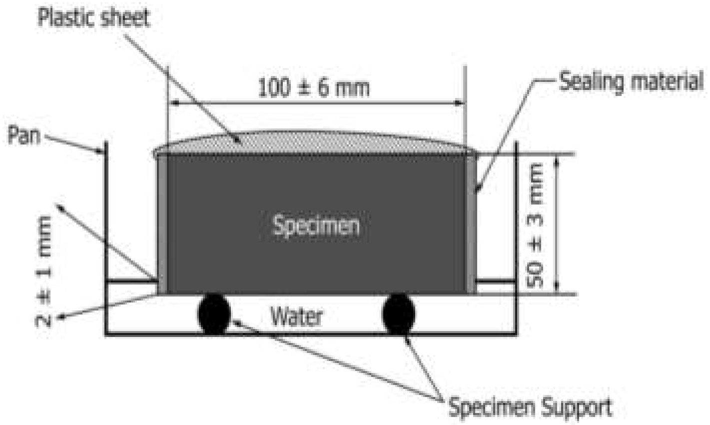


Fig. 3.15: Schematic of Sorptivity Procedure

Measure the mass of the sealed specimen to the nearest 0.01 g and record it as the initial mass for water absorption calculations. Place the support device at the bottom of the pan and fill the pan with tap water so that the water level is 1 to 3 mm above the top of the support device. Maintain the water level 1 to 3 mm above the top of the support device for the duration of the tests. Start the timing device and immediately place the test surface of the specimen on the support device **see Fig. 3.15**. Record the time and date of initial contact with water. Record the mass at the intervals shown in **table 3.11** after first contact with water. The first point shall be at 60 ± 2 s and the second point at 5 min ± 10 s. Subsequent measurements shall be within ± 2 min of 10 min, 20 min, 30 min, and 60 min. The actual time shall be recorded to within ± 10 s continue the measurements every hour, ±5 min, up to 6 h, from the first contact of the specimen with water and record the time within ± 1 min. After the initial 6 h, take measurements once a day up to 3 days, followed by 3 measurements at least 24 h apart during days 4 to 7, take a final measurement that is at least 24 h after the measurement at 7 days. The actual time of measurements shall be recorded within ±1 min. **table 3.14** gives the target times of measurements and the tolerances for the times.

Table 3.14: Time and Tolerances for measurements schedule

Sr. No.	Time(in sec)	Tolerance
1	60 s	2 s
2	5 min	10 s
3	10 min	2 min
4	20 min	2 min
5	30 min	2 min
6	60 min	2 min
7	Every hour up to 6 h	5 min
8	Once a day up to 3 days	2 h
9	Day 4 to 7, 3 measurements 24 h apart	2 h
10	Day 7 to 9,1 measurement	2 h

For each mass determination, remove the test specimen from the pan, stop the timing device if the contact time is less than 10 min, and blot off any surface water with a dampened paper towel or cloth. After blotting to remove excess water, invert the specimen so that the wet surface does not come in contact with the balance pan (to avoid having to dry the balance pan).Within 15 s of removal from the pan, measure the mass to the nearest 0.01 g. Immediately replace the specimen on the support device and restart the timing device.

CHAPTER - 4

RESULTS AND DISCUSSION

4.1 GENERAL

In this chapter results obtained from various tests conducted on HPFRC detailed in chapter 3 are presented and discussed. In order to discuss the results of different parameters like compressive strength, tensile strength, flexure strength and permeability with designation shown in **table 4.1**.Then as per calculated material 24 cubes for different curing conditions (potable and sea water) at 7 days & 28 days were prepared, for investigation of compressive strength and tensile strength. 12 No. beams of size 40 X 40 X 160 mm were casted under different curing conditions i.e. sea water and potable water up to age at testing for investigation of flexure strength and to determine the permeability of high performance fiber reinforced concrete with cylindrical specimens of 100 mm (diameter) X 200 mm (long) were casted under different curing condition i.e. potable water and sea water to age at testing i.e.7 days and 28 days.

Table 4.1: Specimen designation

Sr. No.	Specimen designation	No. of specimens	Property	Age of Testing	Curing Condition
1	Cpw 7	3	Compressive	7	Potable Water
2	Cpw 28	3	Compressive	28	Potable Water
3	Csw 7	3	Compressive	7	Sea Water
4	Csw 28	3	Compressive	28	Sea Water
5	Tpw 7	3	Tensile	7	Potable Water
6	Tpw 28	3	Tensile	28	Potable Water
7	Tsw 7	3	Tensile	7	Sea Water
8	Tsw 28	3	Tensile	28	Sea Water
9	Fpw 7	3	Flexure	7	Potable Water
10	Fpw 28	3	Flexure	28	Potable Water

11	Fsw 7	3	Flexure	7	Sea Water
12	Fsw 28	3	Flexure	28	Sea Water
13	Rpw 7	2	RCPT	7	Potable Water
14	Rpw 28	2	RCPT	28	Potable Water
15	Rsw 7	2	RCPT	7	Sea Water
16	Rsw 28	2	RCPT	28	Sea Water
17	Spw 7	2	Sorptivity	7	Potable Water
18	Spw 28	2	Sorptivity	28	Potable Water
19	Ssw 7	2	Sorptivity	7	Sea Water
20	Ssw 28	2	Sorptivity	28	Sea Water

4.2 COMPRESSIVE STRENGTH

To study the effect on compressive strength, six cubes (size 70.6 x 70.6 x 70.6 mm) each for calculated material weight for both potable water and sea water curing conditions at 7 days & 28 days were cast.

Table 4.2: Comparison between potable water and sea water Curing Conditions for Compressive Strength Tested at 7 and 28 days

Sr. No.	**Compressive Strength after 7 days (N/mm^2)**				**Compressive Strength after 28 days (N/mm^2)**			
	Potable Curing		**Sea water Curing**		**Potable Curing**		**Sea water Curing**	
	Actual	**Average**	**Actual**	**Average**	**Actual**	**Average**	**Actual**	**Average**
1	90.718	88.961	76.386	74.572	107.633	105.597	95.100	93.568
2	89.042		74.741		106.400		94.105	
3	87.124		72.590		102.760		91.500	

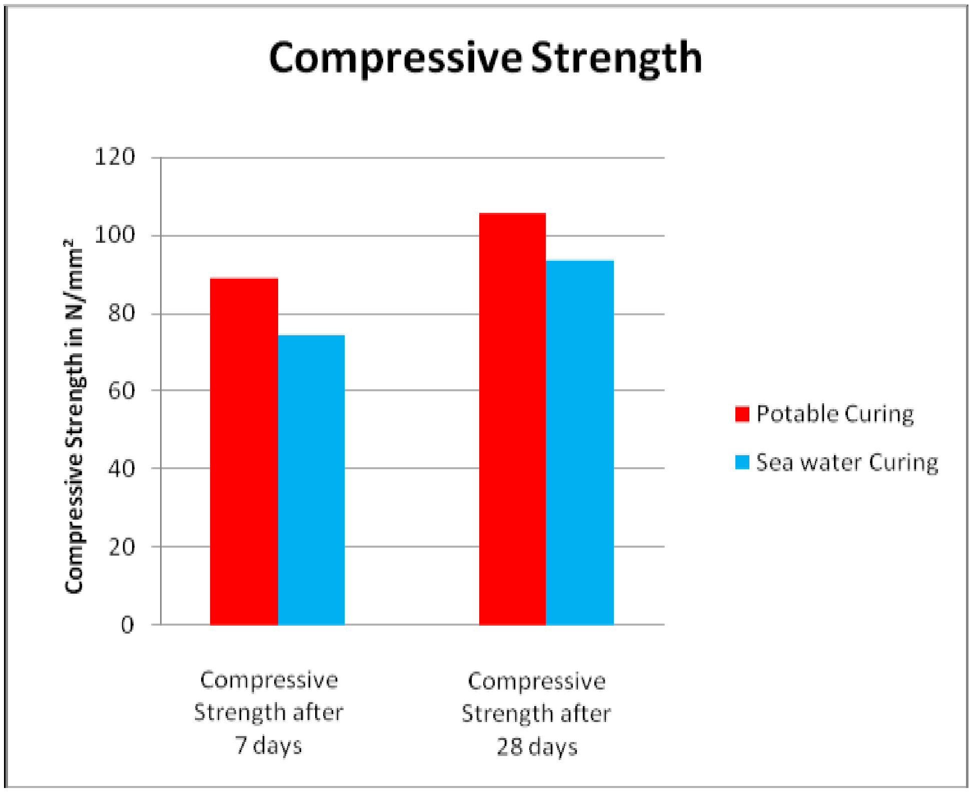


Fig. 4.1: Graph showing Comparison between potable water and sea water curing for Compressive Strength of HPFRC tested after 7 and 28 days

Average results of three specimens of the compressive strength test on HPFRC mix at the age of 7 and 28 days are given in the **table 4.2** and shown graphically in **Fig. 4.1**.

The variation of 7 & 28 days compressive strength of both potable water and sea water curing conditions for mixes under examination are evaluated. It is observed that:-

- The compressive strength of cubes curing with sea water decreases as compared to cubes curing with portable water. This may be due to salt present in sea water. The chemical reactions of seawater on concrete are mainly due to the attack by magnesium sulphate ($MgSO_4$). The mode of attack is crystallization. Potassium and magnesium sulphates (K_2SO_4 and $MgSO_4$) present in salt water can cause sulphate attack on concrete because they can initially react with calcium hydroxide $Ca(OH)_2$, which is present in the set cement formed by the hydration of dicalcium silicate (C_2S) and tricalcium silicate (C_3S). The attack of magnesium sulphate ($MgSO_4$) is particularly damaging.

- The compressive strength of cubes curing with sea water decreases 16.174 % at 7 days as compared to cubes curing with portable water and compressive strength decrease at 28 days 11.39 %, whereas, the reduction at 28 days is comparatively less. This indicates that early strength gain is high as compared with slightly later strength gain.

4.3 TENSILE STRENGTH

To study the effect of tensile strength, 12 cubes (size 70.6 x 70.6 x 70.6 mm) each for calculated material weight for both potable water and sea water curing conditions at 7 days & 28 days were cast and tested as per IS code 5816 : 1999. The load shall be applied without shock and increased continuously at a nominal rate within the range 1.2N/mm^2/min to 2.4N/mm^2/min.

Table 4.3: Comparison between potable water and sea water curing for Tensile Strength Tested at 7 and 28 days

Sr. No.	Tensile Strength after 7 days (N/mm^2)				Tensile Strength after 28 days (N/mm^2)			
	Potable Curing		Sea water Curing		Potable Curing		Sea water Curing	
	Actual	Average	Actual	Average	Actual	Average	Actual	Average
1	5.950	5.894	5.670	5.314	8.947	8.657	8.768	8.392
2	6.274		5.241		8.653		8.386	
3	5.459		5.031		8.370		8,023	

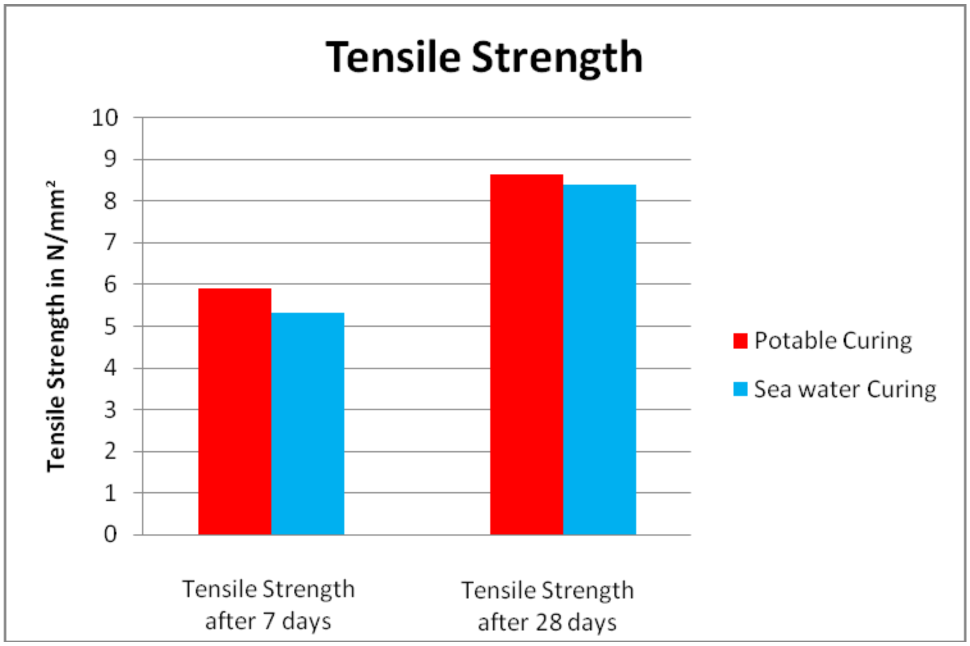


Fig. 4.2: Graph showing Comparison between potable water and sea water curing for Tensile Strength of HPFRC tested after 7 and 28 days

Average results of three specimens of the tensile strength test on HPFRC mix at the age of 7 and 28 days are given in the **table 4.3** and shown graphically in **Fig. 4.2**.

The variation of 7 & 28 days tensile strength of both potable water and sea water curing conditions for mixes under examination are evaluated from table and graphs it is observed that:-

- The tensile strength of cubes curing with sea water decreases as compared to cubes curing with portable water. This may be due to Chloride ions can penetrate into the concrete and cause accelerated corrosion of the reinforcement fiber. The chemical reaction of the cement paste with the high-chloride content of seawater is generally slight and not a primary cause of corrosion. Sodium and potassium ions may produce the alkali aggregate reaction and sulphate and magnesium ions cause a weakening action on the cement paste.

- The tensile strength of cubes curing with sea water decreases 9.84% at 7 days as compared to cubes curing with portable water. Tensile strength decrease at 28 days 3.06 %, whereas, the reduction at 28 days is comparatively less. This indicates that decreases of tensile strength are high at 7 days as compared with decreases of tensile strength 28 days.

4.4 FLEXURAL STRENGTH

To study the effect of flexural strength, six beam (size 40 x 40 x 160 mm) each for calculated material weight for both potable water and sea water curing conditions at 7 days & 28 days casted and tested as per IS code9399: 1979 casted as described **in section 3.4.3 of chapter 3**.

Table 4.4: Comparison between Potable water and sea water Curing Conditions for Flexural Strength Tested at 7 days and 28 days

Sr. No.	Flexural Strength after 7 days (N/mm^2)				Flexural Strength after 28 days (N/mm^2)			
	Potable Curing		Sea water Curing		Potable Curing		Sea water Curing	
	Actual	Average	Actual	Average	Actual	Average	Actual	Average
1	25.889	23.278	23.625	21.890	50.976	45.183	37.195	33.937
2	22.289		21.712		43.411		33.933	
3	21.656		20.334		41,161		30.684	

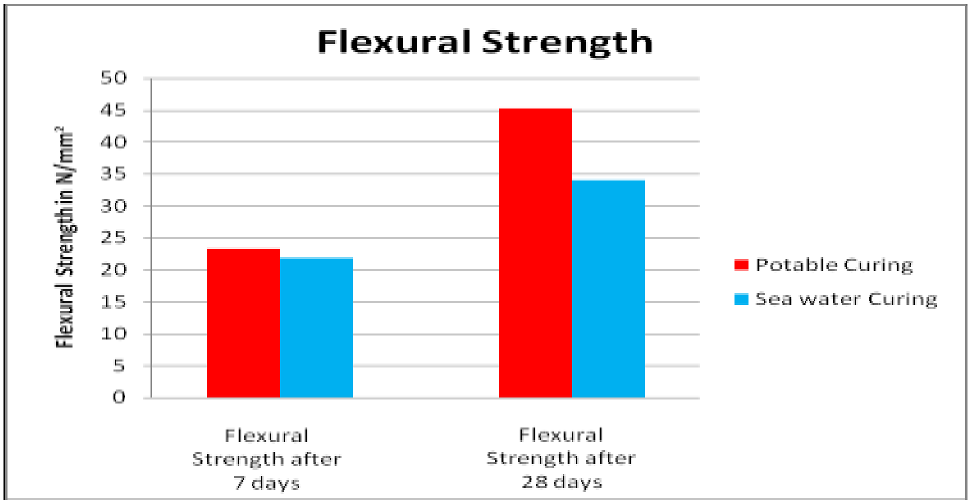


Fig. 4.3: Graph showing Comparison between Potable water and sea water Curing Conditions for Flexural Strength of HPFRC tested after 7 & 28 days

Average results of three specimens of the flexural strength test on HPFRC of at the age of 7 and 28 days are given in the **table 4.4** and shown graphically in **Fig. 4.3.**

The variation of 7 & 28 days flexural strength of both Potable water and Sea water curing conditions under examination are evaluated and from the graphs it is observed that:-

- The flexural strength of specimen curing with sea water also decreases as compared to portable water curing conditions. This may be due to the effect of increases the risk of corrosion of the embedded reinforcing fiber, when the specimen is to be exposed to sea water. The most damaging effect of seawater on concrete structures arises from the action of chlorides on the steel fiber.
- The flexural strength of cubes curing with sea water decreases 5.96% at 7 days as compared to cubes curing with portable water and flexural strength decrease at 28 days 24.89 %, whereas, the reduction at 28 days is high comparatively. This indicates that decreases of flexural strength are high at later age.
- The flexural strength of HPFRC is very high as compared to derived from formula given in IS 456-2000 (flexural strength=0.7 $(f_{ck})^{1/2}$) .Main reason of high flexure strength due to presence of steel fiber in HPFRC. This should be attributed to the fact that the additional steel fibers can bridge cracks and absorb energy which could change the fracture mode of concrete from brittle fracture to plastic fracture and significantly increase the ultimate flexural strength of concrete.

4.5 RAPID CHLORIDE PERMEABILITY TEST

To study the effect of permeability, three cylindrical specimen (size 100mm diameter x 200 mm long) each for calculated material weight for both potable water and sea water curing conditions at 7 days & 28 days were casted and tested as per ASTM C1202-15 . To calculate the permeability by rapid chloride permeability test (RCPT) when the casted cylindrical specimens of 100 X 200 mm attained the age of testing i.e. 7days / 28 days, then cylindrical specimens size of 51 mm (long) x 100 mm (diameter) cut from cores as described in **section 3.4.4** of **chapter 3** with cutter machine as shown in **Fig. 3.10**. The results of RCPT for the specimens under different curing conditions i.e. Potable water Curing and Sea water Curing at the age of 7-days and 28-days are shown in **table 4.5** and graphically in **Fig. 4.4**.

Table 4.5: Results for Rapid Chloride Permeability Test of HPFRC Specimens after 7 days and 28 days

Sr. No.	Total charge passed after 6 hours (Coulombs)				Total charge passed after 6 hours (Coulombs))			
	Potable Curing		Sea water Curing		Potable Curing		Sea water Curing	
	Actual	Average	Actual	Average	Actual	Average	Actual	Average
1	786	833	1151	1366	65	89	401	460
2	815		1434		78		476	
3	898		1512		125		503	

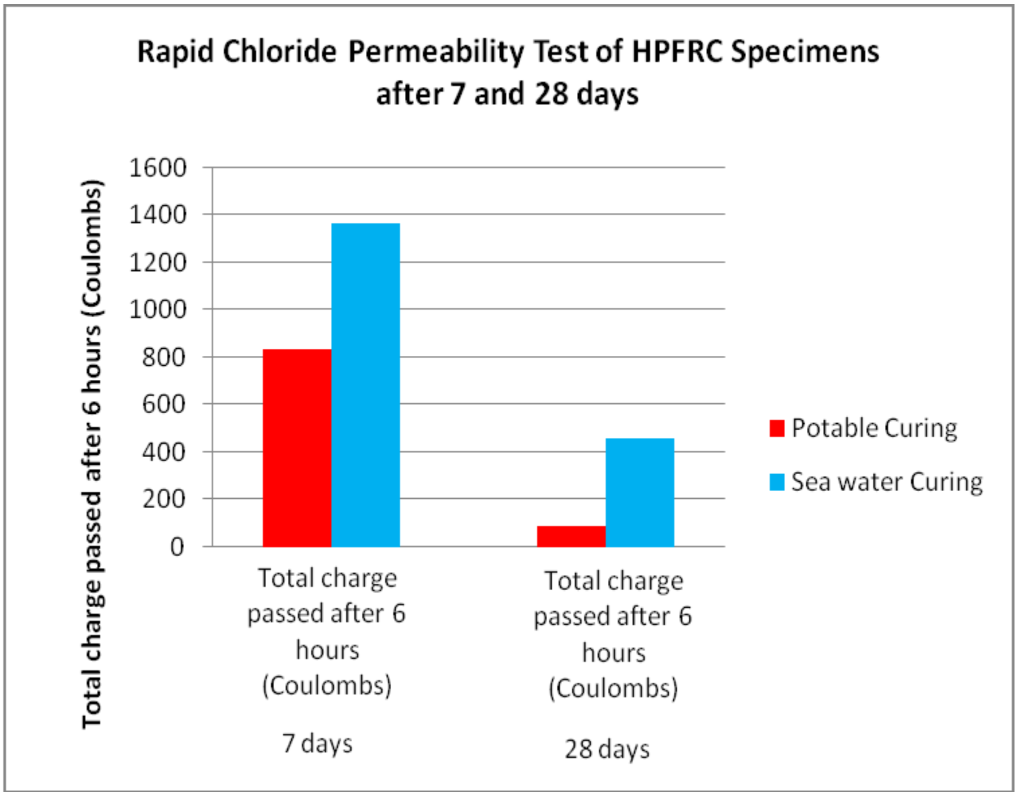


Fig. 4.4: Graph showing comparison for Rapid Chloride Permeability Test of HPFRC tested after 7 and 28 days

From the **table 4.5** and graphically in **Fig.4.4.** It is observed that:-

- In specimen curing with sea water, chloride ions passed were more as compared to specimen curing with potable water for both 7 days as well as 28 curing conditions.
- With potable water curing the water permeability is 5-6 times less in comparison with sea water curing.
- Water permeability at 28 days curing with potable water is negligible and 28 days curing with sea water is very low.

4.6 WATER SSORPTIVITY TEST

To study the effect of Water Sorptivity , three cylindrical specimen (size 100mm diameter x 200 mm long) each for calculated material weight for both potable water and sea water curing conditions at 7 days & 28 days were casted and tested as per ASTM C1585-13 . To calculate the Water Sorptivity by Water Sorptivity Test when the casted cylindrical specimens of 100 X 200 mm attained the age of testing i.e. 28 days, then specimen cut at cutter machine of size 51 mm (long) x 100 mm (diameter). The results of Water Sorptivity for the specimens under different curing conditions i.e. Potable water Curing and Sea water Curing wet at the age of 7-days and 28-days are shown in **table 4.6** and **table 4.7 and** graphically in **Fig.4.5 and Fig.4.6** respectively.

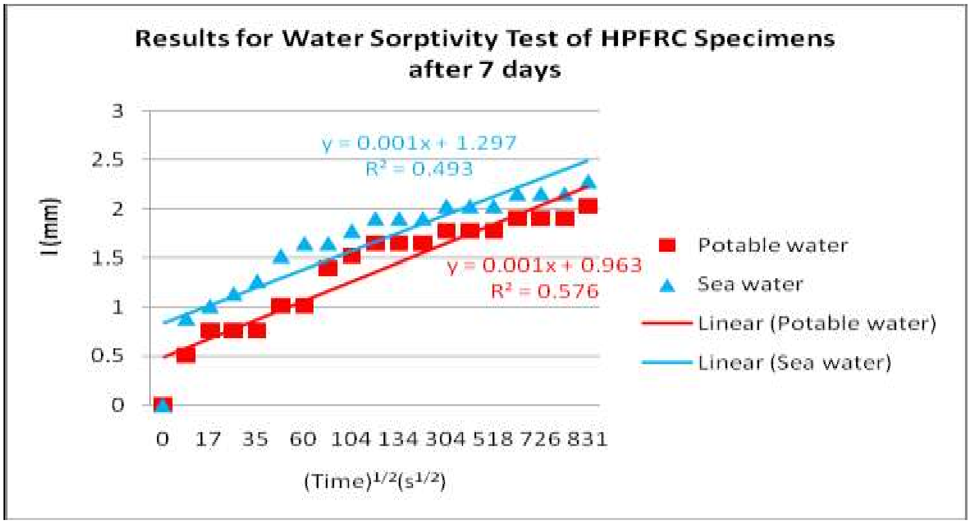


Fig 4.5: Chart for Water Sorptivity Test of HPFRC Specimens after 7 days

Table 4.6: Results for Water Sorptivity Test of HPFRC Specimens after 7 days

Sr. No	Time	(Time)$^{1/2}$ ($s^{1/2}$)	Mass of Specimen (Gms)		Increment in mass(Gms)		Increment in mass/Area/density of water= I	
			potable water	Sea water	potable water	Sea water	potable water	Sea water
1	0	0	982	978	0	0	0	0
2	60	8	986	985	4	7	0.509	0.891
3	300	17	988	986	6	8	0.763	1.018
4	600	24	988	987	6	9	0.763	1.145
5	1200	35	988	988	6	10	0.763	1.273
6	1800	42	990	990	8	12	1.018	1.527
7	3600	60	990	991	8	13	1.018	1.655
8	7200	85	993	991	11	13	1.4	1.655
9	10800	104	994	992	12	14	1.527	1.782
9	14400	120	995	993	13	15	1.655	1.909
10	18000	134	995	993	13	15	1.655	1.909
11	21600	147	995	993	13	15	1.655	1.909
12	92220	304	996	994	14	16	1.782	2.037
13	193200	440	996	994	14	16	1.782	2.037
14	268500	518	996	994	14	16	1.782	2.037
15	432000	657	997	995	15	17	1.909	2.164
16	527580	726	997	995	15	17	1.909	2.164
17	622200	789	997	995	15	17	1.909	2.164
18	691200	831	998	996	16	18	2.037	2.291

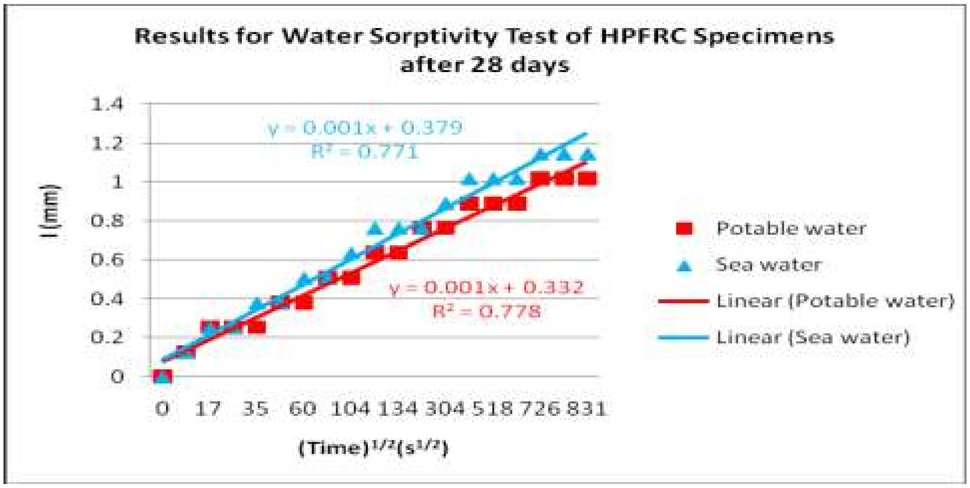


Fig 4.6: Chart for Water Sorptivity Test of HPFRC Specimens after 28 days

Table 4.7: Results for Water Sorptivity Test of HPFRC Specimens after 28 days

Sr. No.	Time	$(\text{Time})^{1/2}$ $(s^{1/2})$	Mass of Specimen (gm)		Increment in mass (gm)		Increment in mass/Area/density of water= I	
			potable water	Sea water	potable water	Sea water	potable water	Sea water
1	0	0	934	946	0	0	0	0
2	60	8	935	947	1	1	0.127	0.127
3	300	17	936	948	2	2	0.254	0.254
4	600	24	936	948	2	2	0.254	0.254
5	1200	35	936	949	2	3	0.254	0.381
6	1800	42	937	949	3	3	0.381	0.381
7	3600	60	937	950	3	4	0.381	0.509
8	7200	85	938	950	4	4	0.509	0.509
9	10800	104	938	951	4	5	0.509	0.636
9	14400	120	939	952	5	6	0.636	0.763
10	18000	134	939	952	5	6	0.636	0.763
11	21600	147	940	952	6	6	0.763	0.763
12	92220	304	940	953	6	7	0.763	0.890
13	193200	440	941	954	7	8	0.890	1.018
14	268500	518	941	954	7	8	0.890	1.018
15	432000	657	941	954	7	8	0.890	1.018
16	527580	726	942	955	8	9	1.018	1.145
17	622200	789	942	955	8	9	1.018	1.145
18	691200	831	942	955	8	9	1.018	1.145

From the **table 4.10 and Fig.4.9 & Fig.4.10** it is observed that:-

- In specimen curing with sea water, Water absorption is more as compared to specimen curing with potable water for both 7 days as well as 28 curing conditions.
- The permeability decreases as the age of specimen increase.

CHAPTER 5

OBSERVATIONS

5.1 GENERAL

The present study was undertaken to investigate the durability properties of high performance fiber reinforced concrete with potable and sea water curing conditions at the age of after 7 and 28 days.

On the basis of present study, following conclusions can be drawn:-

- The compressive strength of cubes curing with sea water decreases as compared to cubes curing with portable water. This may be due to salt present in sea water. The chemical reactions of sea water on concrete are mainly due to the attack by magnesium sulphate ($MgSO_4$). The mode of attack is crystallization. Potassium and magnesium sulphates (K_2SO_4 and $MgSO_4$) present in salt water can cause sulphate attack on concrete because they can initially react with calcium hydroxide $Ca(OH)_2$, which is present in the set cement formed by the hydration of dicalcium silicate (C_2S) and tricalcium silicate (C_3S). The attack of magnesium sulphate ($MgSO_4$) is particularly damaging.
- The compressive strength of cubes curing with sea water decreases 16.174 % at 7 days as compared to cubes curing with portable water and compressive strength decrease at 28 days 11.39 %, whereas, the reduction at 28 days is comparatively less. This indicates that early strength gain is high as compared with slightly later strength gain.
- The tensile strength of cubes curing with sea water decreases as compared to cubes curing with portable water. This may be due to Chloride ions can penetrate into the concrete and cause accelerated corrosion of the reinforcement fiber. The chemical reaction of the cement paste with the high-chloride content of seawater is generally slight and not a primary cause of corrosion. Sodium and potassium ions may produce the alkali aggregate

reaction and sulphate and magnesium ions cause a weakening action on the cement paste.

- The tensile strength of cubes curing with sea water decreases 9.84% at 7 days as compared to cubes curing with portable water. This may be due to salt present in sea water and tensile strength decrease at 28 days 3.06 %, whereas, the reduction at 28 days is comparatively less. This indicates that decreases of tensile strength are high at 7 days as compared with decreases of tensile strength 28 days.
- The flexural strength of specimen curing with sea water also decreases as compared to portable water curing conditions. This may be due to the effect of increases the risk of corrosion of the embedded reinforcing fiber, when the specimen is to be exposed to sea water. The most damaging effect of seawater on concrete structures arises from the action of chlorides on the steel fiber.
- The flexural strength of cubes curing with sea water decreases 5.96% at 7 days as compared to cubes curing with portable water and flexural strength decrease at 28 days 24.89 %, whereas, the reduction at 28 days is high comparatively. This indicates that decreases of flexural strength are high at later age.
- The flexural strength of HPFRC is very high as compared to derived from formula given in IS 456-2000 (flexural strength=0.7 $(f_{ck})^{1/2}$) .Main reason of high flexure strength due to presence of steel fiber in HPFRC. This should be attributed to the fact that the additional steel fibers can bridge cracks and absorb energy which could change the fracture mode of concrete from brittle fracture to plastic fracture and significantly increase the ultimate flexural strength of concrete.
- In specimen curing with sea water, chloride ions passed were more as compared to specimen curing with potable water for both 7 days as well as 28 curing conditions.

- With potable water curing the water permeability is 5-6 times less in comparison with sea water curing.
- Water permeability at 28 days curing with potable water is negligible and 28 days curing with sea water is very low.
- In specimen curing with sea water, Water absorption is more as compared to specimen curing with portable water for both 7 days as well as 28 curing conditions.
- The permeability decreases as the age of specimen increases.

5.2 SCOPE FOR FUTURE WORK

- In the present study 7 and 28 days potable and sea water curing has been considered. The other 7 and 28 days dry and wet curing conditions are need to be study.
- Further study is needed to investigate the micro structure and chemical reaction of HPFRC mix conducting XRD and SEM for different mix compositions of HPFRC.
- Further research is needed for determining the optimum silica fume dosage and cost effectiveness of HPFRC.

CHAPTER 6

REFERENCES

- *AASHTO LRFD Bridge Design Specifications*, Sixth Edition, American Association of State Highway and Transportation Officials, Washington, DC, 2012.
- *AASHTO LRFD Bridge Construction Specifications*, 3rd Edition, American association of State Highway and Transportation Officials, Washington, DC, 2010.
- Adeline, R. and Behloul, M., "High Ductile Beams Without Passive Reinforcement," Proceedings of the Fourth International Symposium on the Utilization of High-Strength/ High-Performance Concrete, 29-31 May 1996, Paris, France, Ed., de Larrard, F. and Lacroix, R., Vol. 3, pp. 1,383–1,390.
- Ay, L., "Curing Tests on Ultra High Strength Plain and Steel Fibrous Cement Based Composites," *Proceedings of the International Symposium on Ultra High Performance*
- *Bétons Fibrés à Ultra-Hautes Performances,* Recommandations Provisoires, (*Ultra High Performance Fiber-Reinforced Concretes)*, Interim Recommendations, SETRA-AFGC, Groupe de travail BFUP, Paris, France, January 2002. (in French and English). Available at http://www.afgc.asso.fr/statuts-de-lassociation-francaise-de-genie-civil.html [Cited November 23, 2011].
- Blaise, P.Y. and Couture, M., "Precast, Prestressed Pedestrian Bridge—World's First Reactive Powder Concrete Structure," *PCI Journal*, Vol. 44, No. 5, September/ October 1999, pp. 60–71.
- Boulekbache B, Hamrat M, Chemrouk M, Amziane S. Flowability of fibre-reinforced concrete and its effect on the mechanical properties of the material.
- Constr Build Mater 2010;24:1664–71.

- *Concrete*, Ed., Schmidt, M., Fehling, E., and Geisenhanslüke, C., Kassel University Press, Kassel, Germany, 2004, pp. 695–701.
- Collepardi, S. et al., "Mechanical Properties of Modified Reactive Powder Concrete," *Proceedings of the Fifth CANMET/ACI International Conference on Superplasticizers and Other Chemical Admixtures in Concrete*, 1997, Rome, Italy, Publication No. SP-173, Ed., Malhotra V.M., American Concrete Institute, Farmington Hills, MI, pp. 1–21.
- Coppola, L. et al., "Influence of Super-Plasticizer Type on Compressive Strength of Reactive Powder Mortars," *Proceedings of the Fifth CANMET/ACI International Conference on Superplasticizers and Other Chemical Admixtures in Concrete*, 1997, Rome, Italy, Publication No. SP-173, Ed., Malhotra V.M., American Concrete Institute, Farmington Hills, MI, pp. 537–557.
- Endicott, W.A., "A Whole New Cast," *ASPIRE*, Summer 2007, pp. 26–29. Available at http://www.aspirebridge.org [Cited November 23, 2011].
- Graybeal, B., "Material Property Characterization of Ultra-High Performance Concrete," FHWA, U.S. Department of Transportation, Report No. FHWA-HRT-06-103, McLean, VA, 2006.
- Graybeal, B., "Ultra-High Performance Concrete," *TechNote*, FHWA-HRT-11-038, Federal Highway Administration, McLean, VA, 2011.
- Graybeal, B., Perry, V., and Royce, M., "UHPC Ultra-High Performance Concrete," NHI Innovations Webinar, November 18, 2010. Available at https://connectdot.connectsolutions.com/n13408320101[Cited April 3, 2012].
- Gowripalan, N. and Gilbert, R.I., "Design Guidelines for Ductal Prestressed Concrete Beams," The University of New South Wales, Sydney, Australia, 2000.

- Graybeal, B., “Construction of Field-Cast Ultra-High Performance Concrete Connections,” *TechNote*, Federal Highway Administration, McLean, VA, FHWA-HRT-12-038, 2012.
- Graybeal, B. and Stone, B., “Compression Response of a Rapid-Strengthening Ultra-High Performance Concrete Formulation,” FHWA, U.S. Department of Transportation, Report No. FHWA-HRT-12-065, National Technical Information Service Accession No. PB2012-112545, 2012.
- Graybeal, B. and Davis, M., “Cylinder or Cube: Strength Testing of 80 to 200 MPa (11.6 to 29 ksi) Ultra-High-Performance Fiber-Reinforced Concrete,” *ACI Materials Journal*, Vol. 105, No. 6, November–December 2008, pp. 603–609.

- Graybeal, B. and Davis, M., “Cylinder or Cube: Strength Testing of 80 to 200 MPa (11.6 to 29 ksi) Ultra-High-Performance Fiber-Reinforced Concrete,” *ACI Materials Journal*, Vol. 105, No. 6, November–December 2008, pp. 603–609.

- Graybeal, B., “Structural Behavior of Ultra-High Performance Concrete Prestressed IGirders,” FHWA, U.S. Department of Transportation, Report No. FHWA-HRT-06-115, 2006. Available through National Technical Information Service at http://www.ntis.gov with Accession No. PB2007104386 [Cited November 23, 2011].
- Habel, K. et al., “Ultra-High Performance Fiber Reinforced Concrete Mix Design in Central Canada,” *Canadian Journal of Civil Engineering*, Vol. 35, No. 2, February 2008, pp. 217–224.
- Holschemacher, K. and Weiße, D., “Economic Mix Design Ultra High-Strength Concrete,” *Seventh International Symposium on the Utilization of High-Strength/High-Performance Concrete*, Vol. II, Publication No. SP-228, Ed., Russell, H.G., American Concrete Institute, Farmington Hills, MI, 2005, pp. 1,133–1,144.

- Heinz, D. and Ludwig, H.-M., “Heat Treatment and the Risk of DEF Delayed Ettringite Formation in UHPC,” *Proceedings of the International Symposium on Ultra High Performance Concrete*, Ed., Schmidt, M., Fehling, E., and Geisenhanslüke, C., Kassel University Press, Kassel, Germany, 2004, pp. 717–730.
- Heinz, D., Urbonas, L., and Gerlicher, T., “Effect of Heat Treatment Method on the Properties of UHPC,” *Proceedings of Hipermat 2012 3rd International Symposium on UHPC and Nanotechnology for High Performance Construction Materials*, Ed., Schmidt, M., Fehling, E., Glotzbach, C., Fröhlich, S., and Piotrowski, S., Kassel University Press, Kassel, Germany, 2012, pp. 283–290.
- Japan Society of Civil Engineers (JSCE), *Recommendations for Design and Construction of Ultra High Strength Fiber Reinforced Concrete Structures (Draft)*, JSCE Guidelines for Concrete No. 9, 2006.
- Kim, B.-S. et al., “R&D Activities and Application of Ultra High Performance Concrete to Cable-Stayed Bridges,” *Proceedings of Hipermat 2012 3rd International Symposium on UHPC and Nanotechnology for High Performance Construction Materials*, Ed., Schmidt, M., Fehling, E., Glotzbach, C., Fröhlich, S., and Piotrowski, S., Kassel University Press, Kassel, Germany, 2012, pp. 865–872.
- Kazemi, S. and Lubell, A., “Influence of Specimen Size and Fiber Content on Mechanical Properties of Ultra-High-Performance Fiber-Reinforced Concrete,” *ACI Materials Journal*, Vol. 109, No. 6, November–December 2012, pp. 675–684.
- Kim, S.W. et al., “Effect of Filling Method on Fiber Orientation & Dispersion and Mechanical Properties of UHPC,” *Proceedings of the Second International Symposium on Ultra High Performance Concrete*, Ed., Fehling,

E., Schmidt, M., and Stürwald. S., Kassel University Press, Kassel, Germany, 2008, pp. 185–192.

- Keierleber, B. et al., "FHWA, Iowa Optimize Pi Girder," *ASPIRE*, Winter 2010, pp. 24–26. Available at http://www.aspirebridge.org [Cited November 23, 2011].
- Keierleber, B. et al., "Design of Buchanan County, Iowa, Bridge, Using Ultra High- Performance Concrete and PI-Beam Cross Section," *Proceedings of the PCI National Bridge Conference*, October 4–7, 2008, Orlando, FL, Compact Disc, Paper 27.
- Leonhardt, S., Lowke, D., and Gehlen, G., "Effect of Fibers on Impact Resistance of Ultra High Performance Concrete," *Proceedings of Hipermat 2012 3rd International Symposium on UHPC and Nanotechnology for High Performance Construction Materials*, Ed., Schmidt, M., Fehling, E., Glotzbach, C., Fröhlich, S., and Piotrowski, S., Kassel University Press, Kassel, Germany, 2012, pp. 811–817.
- Leutbecher, T. and Fehling, E., "Structural Behaviour of UHPC Under Tensile Stress and Biaxial Loading," *Proceedings of the International Symposium on Ultra High PerformanceConcrete*, Ed., Schmidt, M., Fehling, E., and Geisenhanslüke, C., Kassel University Press, Kassel, Germany, 2004, pp. 435–446.
- Mazanec, O. and Schieβl, P., "Mixing Time Optimization for UHPC," *Proceedings of the Second International Symposium on Ultra High Performance Concrete*, Ed., Fehling, E., Schmidt, M., and Stürwald. S., Kassel University Press, Kassel, Germany, 2008, pp. 401–408.
- Maguire, M. et al., "Ultra-High-Performance Concrete in Standard Precast/Prestressed Concrete Products," Proceedings of the PCI National Bridge Conference, September 12–15, 2009, San Antonio, TX, Paper 33.

- Markovic I. High-performance hybrid-fibre concrete – development and utilisation. Technische Universität Delft, Ph.D. thesis; 2006.
- Massidda, L. et al., "High Pressure Steam Curing of Reactive-Powder Mortars," *Fifth CANMET/ACI Conference on Recent Advances in Concrete Technology, Fifth International Conference*, Publication No. SP-200, Ed., Malhotra, V.M., American Concrete Institute, Farmington Hills, MI, 2001, pp. 447–463.
- Mazanec, O., Lowke, D., and Schieβl, P., "Mixing of High Performance Concrete: Effect of Concrete Composition and Mixing Intensity on Mixing Time," *Materials and Structure*, Vol. 43, No. 3, 2010, pp. 357–365.
- Meade, T.M. and Graybeal, B.A., "Flexural Response of Lightly Reinforced Ultra-High Performance Concrete Beams," Proceedings of the Third International fib Congress and Exhibition Incorporating the PCI Annual Convention and National Bridge Conference, Washington, DC, May 29–June 2, 2010, Compact Disc, Paper 101.
- Ozyildirim, C., "Evaluation of Ultra-High-Performance Fiber-Reinforced Concrete," Virginia Center for Transportation Innovation and Research, Report No. FHWA/VCTIR 12-R1, Federal Highway Administration, McLean, VA, 2011.
- Orgass, M. and Klug, Y., "Fiber Reinforced Ultra-High Strength Concretes," Proceedings of the International Symposium on Ultra High Performance Concrete, Ed., Schmidt, M., Fehling, E., and Geisenhanslüke, C., Kassel University Press, Kassel, Germany, 2004, pp. 637–647.
- Piotrowski, S. and Schmidt, M., "Life Cycle Cost Analysis of a UHPC-Bridge on Example of Two Bridge Refurbishment Designs," *Proceedings of Hipermat 2012 3rd International Symposium on UHPC and Nanotechnology for High Performance Construction Materials*, Ed., Schmidt, M., Fehling, E.,

Glotzbach, C., Fröhlich, S., and Piotrowski, S., Kassel University Press, Kassel, Germany, 2012, pp. 957–964.

- Plank, J., Schröfl, C., and Gruber, M., "Use of a Supplemental Agent to Improve Flowability of Ultra-High-Performance Concrete," *Ninth ACI International Conference on Superplasticizers and Other Chemical Admixtures*, Publication No. SP-262, Ed., Gupta, P., Holland, T.C., and Malhotra, V.M., American Concrete Institute, Farmington Hills, MI, 2009, pp. 1–16.
- Qian, S. and Li, V.C., "Simplified Inverse Method for Determining the Tensile Properties of SHCCs," *Journal of Advanced Concrete Technology*, Vol. 6, No. 2, 2008, pp. 353–363.
- Rossi, P., "Ultra-High-Performance Fiber-Reinforced Concretes. A French Perspective on Approaches Used to Produce High-Strength, Ductile Fiber-Reinforced Concrete," *Concrete International*, Vol. 23, No. 12, December 2001, pp. 46–52.
- Rossi, P. et al., "Bending and Compressive Behaviors of a New Cement Composite," *Cement and Concrete Research*, Vol. 35, No. 1, 2005, pp. 27–33.
- R. Yu , P. Spiesz, H.J.H. Brouwers Effect of nano-silica on the hydration and microstructure development of Ultra-High Performance Concrete (UHPC) with a low binder amount Department of the Built Environment, Eindhoven University of Technology, P.O. Box 513, 5600 MB Eindhoven, The Netherlands.
- Schmidt, M., "Sustainable Building With Ultra-High-Performance Concrete (UHPC)— Coordinated Research Program in Germany," *Proceedings of Hipermat 2012 3rd International Symposium on UHPC and Nanotechnology for High Performance Construction Materials*, Ed., Schmidt, M., Fehling, E., Glotzbach, C., Fröhlich, S., and Piotrowski, S., Kassel University Press, Kassel, Germany, 2012, pp. 17–25.

- Schmidt, M. et al., "Mix Design and Properties of Ultra-High Performance Fiber Reinforced Concrete for the Construction of a Composite Concrete UHPFRC-Concrete Bridge," *Proceedings of IABSE Symposium Improving Infrastructure Worldwide—Bringing People Closer*, IABSE, Weimar, Germany, Vol. 93, 2007, pp. 466–476. Also *Ultra HighPerformance Concrete (UHPC)—10 Years of Research and Development at the University of Kassel*, Ed., Schmidt, M. and Fehling, E., Kassel University Press, Kassel, Germany, 2007, pp. 44–54.
- Stiel, T., Karihaloo, B., and Fehling, E., "Effect of Casting Direction on the Mechanical Properties of CARDIFRC®," *Proceedings of the International Symposium on Ultra High Performance Concrete*, Ed., Schmidt, M., Fehling, E., and Geisenhanslüke, C., Kassel University Press, Kassel, Germany, 2004, pp. 481–493.
- Schachinger, I., Hilbig, H., and Stengal, T., "Effect of Curing Temperatures at an Early Age on the Long-Term Strength Development of UHPC," *Proceedings of the Second* Teichmann, T. and Schmidt, M., "Mix Design and Durability of Ultra High Performance Concrete (UHPC)," *Proceedings of the 4th International Ph.D. Symposium in Civil Engineering*, Munich, 2002, pp. 341–347.
- Schmidt, M. and Fröhlich, S., "Testing of UHPC," Proceedings of the Third International fib Congress and Exhibition Incorporating the PCI Annual Convention and National Bridge Conference, Washington, DC, May 29–June 2, 2010, Compact Disc, Paper 237.
- Senff L, Hotza D, Lucas S, Ferreira VM, Labrincha JA. Effect of nano-SiO2 and nano-TiO2 addition on the rheological behavior and the hardened properties of cement mortars. Mater Sci Eng A 2012;532:354–61.
- Skazlić, M., Serdar, M., and Bjegović D., "Influence of Test Specimens Geometry on Compressive Strength of Ultra High Performance Concrete,"

Proceedings of the Second International Symposium on Ultra High Performance Concrete, Ed., Fehling, E., Schmidt, M., and Stürwald. S., Kassel University Press, Kassel, Germany, 2008, pp. 295–301.

- Stürwald, S. and Fehling, E., "Design of Reinforced UHPFRC in Flexure," Proceedings of Hipermat 2012 3rd International Symposium on UHPC and Nanotechnology for High Performance Construction Materials, Ed., Schmidt, M., Fehling, E., Glotzbach, C., Fröhlich, S., and Piotrowski, S., Kassel University Press, Kassel, Germany, 2012, pp. 443–450.
- Visage, E.T. et al., "Experimental and Analytical Analysis of the Flexural Behavior of UHPC Beams," Proceedings of Hipermat 2012 3rd International Symposium on UHPC and Nanotechnology for High Performance Construction Materials, Ed., Schmidt, M., Fehling, E., Glotzbach, C., Fröhlich, S., and Piotrowski, S., Kassel University Press, Kassel, Germany, 2012, pp. 403–410.
- Wille, K., Naaman, A.E., and El-Tawil, S., "Optimizing Ultra-High-Performance Fiber-Reinforced Concrete," Concrete International, Vol. 33, No. 9, September 2011, pp. 35–41.
- Wille, K. and Parra - Montesinos, G., "Effect of Beam Size, Casting Method, and Support Conditions on Flexural Behavior of Ultra-High-Performance Fiber-Reinforced Concrete," *ACI Materials Journal*, Vol. 109, No. 3, May–June 2012, pp. 379–388.