MECHANICAL PROPERTIES OF ULTRA HIGH STRENGTH FIBER REINFORCED CONCRETE

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MECHANICAL PROPERTIES OF ULTRA HIGH STRENGTH FIBER REINFORCED CONCRETE

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ABSTRACT

The usage of Ultra-High Strength Fiber Reinforced Concrete (UHSFRC) with higher compressive strength (15,000-29,000 psi) in construction industry has been increasing worldwide. UHSFRC is also known as reactive powder concrete (RPC) which exhibits excellent durability and mechanical properties. This is one of the latest and emerging topics in the concrete technology. Structural elements cast with UHPC can carry larger loads and exhibit energy absorption capacity with smaller sections.

The high compressive strength, higher tensile strength along with almost negligible water and chloride permeability therefore better durability of this new concrete material makes it UHSFRC. The basic principle in UHSFRC is to make the cement matrix as dense as possible, by reducing the micro cracks and capillary pores in the concrete and also to make a dense transition zone between cement matrix and aggregates. These special properties of concrete can be achieved by eliminating the coarse aggregates and replacing them with quartz sand of maximum size of 600 microns.

Concrete (UHSFRC) using materials that are available locally are always economical since the patented products are very expensive and the materials such as silica sand and quartz powder are not readily available. The research also includes use of recognized mineral admixtures, natural river sand, steel fibers, and superplasticizers (Sika Viscocrete 2100 – 3% by weight of cement, Melflux 4930 – 1% by weight of cementitious material) and an optimum dosage of silica fume was 15 % by weight of the cement
The use of UHPC in the construction of shear keys can be a good solution for achieving long lasting bridge systems.

The fresh and hardened mechanical properties of the UHSFRC were studied such as workability of the mix, compression test on cubes, split tensile test on cylinders, flexural tensile test for both reinforced and unreinforced concrete beams, rebar pull-out tests, impact test on panels and testing for shear keys.

Two different curing practices were used in this work: Moist Curing (MC) and Heat Curing (HC). Two different types of cements used were ASTM Type I and Type III cements. Type I cement is commonly used in all the construction works whereas Type III cement is used in special applications where early high strengths are required. Both the cements are used for the comparative study, keeping all the proportions constant.

Compressive strength of 21,500 psi was achieved with concrete made of type III cement using moist curing practice. Split tensile strength of 2,300 psi and flexural strength of 3,300 psi were gained using Type III cement and moist curing practice. Highest compressive strength of 28,150 psi is achieved using heat curing practice. It was found that heat curing practice may be artificially inflating the compressive strength. The split tensile strength, and flexural strength results of heat-cured specimens have lower strength compared to moist cured specimens. The moment capacity of the fiber reinforced concrete is twice than the conventional concrete, due to the denser microstructure, absence of coarse aggregates, and cement silica reaction. Potential application of UHSFRC in shear keys of adjacent box beam girder bridges was demonstrated on small joint test specimens with sand blasted surface. It is concluded that the use of the mix design developed in this study for UHSFRC is feasible for such box beam bridges.
ACKNOWLEDGEMENTS

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I would like to acknowledge Jay Whitt from ESSROC for his immense technical support and providing Type III cement. I would like to acknowledge P.T. Hutchins Company for proving me powdered Superplasticizer. I would also like to acknowledge ELKEM for providing me Un-densified Silica Fume throughout my research work. I would Also like to thank Gary Neilson from Sika Corporation for proving High range Water Reducing Agent , I would like to extend my sincere thanks to Mr. David McVaney for his help in the laboratory. I would like thank my family for supporting me. Very special thanks to my colleagues; Abdisa Musa, Srikanth Marchetty, Sunil Gowda, Dheeraj, Ali, Haboubh Mohammed for their help in laboratory in casting and testing the samples.
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<tr>
<td>$f'_c$</td>
<td>Compressive strength of concrete in MPA or psi or ksi</td>
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<tr>
<td>ACI</td>
<td>American Concrete Institute</td>
</tr>
<tr>
<td>MPA</td>
<td>Megapascal (N/mm$^2$)</td>
</tr>
<tr>
<td>UHPFRC</td>
<td>Ultra High Performance Fiber Reinforced Concrete</td>
</tr>
<tr>
<td>RPC</td>
<td>Reactive powder concrete</td>
</tr>
<tr>
<td>DSP</td>
<td>Densified with small particles</td>
</tr>
<tr>
<td>VHSC</td>
<td>Very high strength concrete</td>
</tr>
<tr>
<td>UHSC</td>
<td>Ultra High Strength Concrete</td>
</tr>
<tr>
<td>SHSC</td>
<td>Super High Strength Concrete</td>
</tr>
<tr>
<td>MDF</td>
<td>Macro Defect Free Concrete</td>
</tr>
<tr>
<td>HPC</td>
<td>High Performance Concrete</td>
</tr>
<tr>
<td>SFCBC</td>
<td>Steel fibrous cement based composite</td>
</tr>
<tr>
<td>MSFRC</td>
<td>Multi Scale Fiber Reinforced Concrete</td>
</tr>
<tr>
<td>UHPC</td>
<td>Ultra High Performance Concrete</td>
</tr>
<tr>
<td>$\mu$</td>
<td>Microns</td>
</tr>
<tr>
<td>W/c</td>
<td>water to cement ratio</td>
</tr>
<tr>
<td>W/cm</td>
<td>water to cementitious materials ratio</td>
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<tr>
<td>WRA</td>
<td>Water Reducing Agent</td>
</tr>
<tr>
<td>HRWRA</td>
<td>High Range Water Reducing Agent</td>
</tr>
<tr>
<td>ITZ</td>
<td>Interfacial transition zone</td>
</tr>
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<tr>
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<td>Normal Strength Concrete</td>
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<td>Fiber Reinforced Concrete</td>
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<td>GFRP</td>
<td>Glass Fiber Reinforced Plastic</td>
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<td>CETE</td>
<td>Centre d’Etudes Techniques de l’Equipement</td>
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<td>SETRA</td>
<td>Service d’Etudes Techniques des Routes et AutoRoute</td>
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<td>Calcium Hydroxide</td>
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<td>American Society of Testing and Materials</td>
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<td>FHWA</td>
<td>Federal Highway Administration</td>
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<tr>
<td>AASHTO</td>
<td>American Association of State Highway and Transportation Officials</td>
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<tr>
<td>TT</td>
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<tr>
<td>DTT</td>
<td>Delayed Thermal Treatment</td>
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<td>DDTT</td>
<td>Doubly Delayed Thermal Treatment</td>
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<td>( w_c )</td>
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<td>( E_c )</td>
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<td>( f'_{ATT} )</td>
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<td>Sand Blasted</td>
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<tr>
<td>BSI</td>
<td>Beton Special Industrial</td>
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<td>AFGC</td>
<td>Association Française de Génie Civil</td>
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CHAPTER I
INTRODUCTION

1.1 General Background

In the world of construction, concrete is one of the most widely used material for centuries. Concrete is absolutely indispensable in all construction works such as roads, high rise buildings, walkways, dams, bridges and many more applications. It is estimated that the consumption of concrete is 12 billion tons per year all over the world. Humans consumes no other material in greater quantities than concrete with the exception of water. Some of the industry experts feel like “concrete is chemistry”. This statement is absolutely true due to the increasing development of admixtures which chemically affect the properties of concrete. Concrete helps for the growth of civilization, can be fabricated on site, and requires less labor efforts. Concrete is widely used construction material around the globe and is evolving day by day. In the last decade there has been significant improvement in the concrete which today is providing innovative solutions, with regard to design, placement, and aesthetic value. Concrete has changed in the 20th century due to the developments in mineral and organic chemistry and improvement in production techniques.

We have been using Portland cement as a binding material for the past 200 years. There are some concrete harbors in Italy which are still functioning even after 200 years.
Meanwhile a modern day concrete structure is likely to last for 50 years subjected to harsh environment. In 1950s 5100 Psi (35 Mpa) was considered to be the high strength concrete, whereas in 1960s concrete with strength 41 to 55 Mpa (6,000 to 7,900 Psi) was used and 1970s 9,400 Psi (65 Mpa) concrete was considered and used in the construction of high rise buildings. Most recently concrete with compressive strength greater than 120 Mpa (17,500 Psi) is developed and called as ultra-high performance concrete. This innovation led to build economical sections epically for high rise buildings.

Ultra-high performance concrete is purely new class of concrete and has good mechanical properties, durability and long-term stability compared to high performance concrete (HPC). Ultra-high performance concrete is made by using very fine aggregates, low water cement ratio and high range water reducing agents to make the concrete flowable.

The researchers from USA and Europe came up with the recipe of the romans concrete after investigating the harbors. “Romans made concrete by mixing lime and volcanic rock. For underwater structures lime and volcanic ash were mixed to form mortar, and this mortar and volcanic tuff were packed into the wooden forms. The sea water instantly triggered a hot chemical reaction.

Concrete industry has been eagerly looking for an innovation in the concrete which has high strength as well as durable, for the construction of taller buildings and rehabilitation of structures. High rise structures are usually densely reinforced and have very little room for concrete to be filled. Which can be a major defect if not filled and compacted completely. Ultra-high performance concrete is self-compacting and easily fills up the spaces between the reinforcing bars.
Ultra-high performance concrete (UHPC) became commercially available in the United States in the year 2000. The Federal Highway Administration (FHWA) started investigating UHPC for highway infrastructure since 2001. The work led to the use of UHPC in several bridge applications, including precast, pre-stressed girders; precast waffle panel for bridge decks; and as a joint material between precast concrete deck panels and girders and between the flanges of adjacent girders.

1.2 Statement of the Problem

Ultra-high performance concrete is a patented product which possesses superior mechanical properties compared to conventional concrete, but the main concern is that it is not cost effective. The cost is relatively 20 times expensive than conventional concrete. The existing options for UHPC are limited and expensive due to use of silica sand (Ottawa sand) and quartz powder which are not readily available. Traditionally UHPC is produced with the sand having defined particle size distribution. The main reason for considering the Ottawa sand is due to its mineralogy, particle shape and size distribution. Ottawa sand is siliceous almost rounded to sub rounded grains and have the smooth surface. This properties of the Ottawa sand are very favorable when we work with the lower water to cement ratios by enhancing the workability. The conventional sand used in this research is siliceous river sand with sub rounded grain size.

As a result of extensive research globally for the past few years production of UHPC is no longer a patented concrete product. Taking notes of research work pertaining to utilization of naturally available materials for producing UHPC, an attempt is made to produce UHPC using materials available locally. The main objective of this research is to use the naturally available materials.
1.3 Research Objectives

The main objective of this research work is to produce Ultra high performance fiber reinforced concrete using naturally available materials.

- To obtain the design mix for Ultra High Strength Fiber Reinforced Concrete (UHSFRC)
- To study the mechanical properties such as Compressive strength, split tensile strength, flexural tensile strength.
- To study the effect of different High Range Water Reducing Agents
- To study the effect of different curing practices on mechanical properties of the concrete
- To study the structural behavior of UHPC
- To apply the UHPC material in the shear keys

1.4 Scope of Thesis

This thesis is organized into 5 Chapters. Chapter 1 gives the general background, statement of the problem, research objectives, and scope of thesis. In Chapter 2, Literature review about the Ultra-High Performance Concrete, Fiber Reinforced Concrete, Large scale applications, different mix proportions from various researchers and mechanical properties such as compressive strength, split tensile strength, flexural strength, and application of UHPC in shear keys. Chapter 3 explains about the experimental program such as materials used, preparation of samples, mixing procedure and curing procedure, different super plasticizers used. Chapter 4 presents laboratory results and discussion. Chapter 5 describes conclusions and recommendations for the future work.
2.1 Introduction

Ultra-High Strength Fiber Reinforced Concrete is a new type of material which is no different to conventional concrete, except the fact that coarse aggregates are not used. Maximum size of the aggregates ranges from 150 to 600 µ.

2.1.1 General definition of Ultra-High Performance Concrete (UHPC)

According to American Concrete Institute (ACI), concrete meeting special combinations, performance and uniformity requirements that cannot always be achieved routinely by using conventional constituents, normal mixing and curing practices ACI does not have the definition for Ultra High performance concrete (1). The US department of transportation describes UHPC as the material which has compressive strength over 150 Mpa, internal fiber reinforcement (for ductile behavior) and high cement content. It has no aggregates except, sand resulting in low porosity and consequently high durability. UHPC has less water to cement ratio and high range water reducing agents to make the concrete workable (2).
Other names of Ultra high performance concrete are (3)

- Ultra-high performance fiber reinforced concrete (UHPFRC)
- Reactive powder concrete (RPC)
- Densified with small particles (DSP) concrete
- Very high strength concrete (VHSC)
- Ultra-high strength concrete
- Super high strength concrete
- Macro defect free (MDF)
- Fiber-reinforced high-performance concrete (HPC)
- Steel fibrous cement-based composite (SFCBC)
- Multi-scale fiber-reinforced concrete (MSFRC)

UHPC exhibits excellent mechanical and durability properties which is entirely a new product in the concrete industry. High compressive strength (greater than 150 Mpa (21,750 Psi)), high tensile strength (>7 Mpa (1045 Psi)), flexural strength (40 Mpa (5800 Psi)), young’s modulus of 47 GPA, ductility and almost negligible chloride penetration. The basic principle in UHPC is to achieve the dense matrix as much as possible by reducing the micro cracks and capillary pores in cement matrix and to get a dense transition zone between cement matrix and aggregates. All these requirements of UHPC can be achieved by entirely eliminating the coarse aggregates and using the quartz sand of maximum grain size 0.6 mm. (600µ). The properties further can also be improved by adding silica fume about 10 to 15 % by weight of the cement, which reacts with calcium hydrate to form calcium silicate hydrate which gives the additional strength to the cement matrix.
As the water cement ratio increases the strength of the concrete tend to decrease, so lesser water cement ratio (less than 0.25) is generally used and to produce good workable concrete high range water reducing agents are been used. Ductility can be achieved by adding different fibers at different dosages (4). Generally high volume fraction steel fiber dosage is used for Ultra High Performance Concrete which is more than 2% by volume. Figure 1 shows evaluation of concrete over the years and Figure 2 shows strength development of concrete over 100 years.

Figure 1 - Evolution of concrete over the years (36)

Figure 2 - Concrete strength development over 100 years (5)
Table 1 shows the difference between Normal Concrete, High Strength Concrete, Very High Strength Concrete, and Ultra High Strength Concrete.

**Table 1 – Different Types of Concretes (36)**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Conventional concrete</th>
<th>High-strength concrete</th>
<th>Very-high-strength concrete</th>
<th>Ultra-high-strength concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strength, Mpa(ksi)</td>
<td>&lt;50 (7250)</td>
<td>50-100 (7250-14,500)</td>
<td>100-150 (14,500-21,750)</td>
<td>&gt;150(21,750)</td>
</tr>
<tr>
<td>Water/cement ratio</td>
<td>&gt;0.45</td>
<td>0.45-0.3</td>
<td>0.3-0.25</td>
<td>&lt;0.25</td>
</tr>
<tr>
<td>Chemical admixtures</td>
<td>Not necessary</td>
<td>WRA*/HRWR°</td>
<td>HRWR</td>
<td>HRWR</td>
</tr>
<tr>
<td>Mineral admixtures</td>
<td>Not necessary</td>
<td>Fly ash</td>
<td>Silica fume/ fly ash</td>
<td>Silica fume /fly ash</td>
</tr>
<tr>
<td>Permeability coefficient (cm/s)</td>
<td>&gt;10^{-10}</td>
<td>10^{-11}</td>
<td>10^{-12}</td>
<td>&lt;10^{-13}</td>
</tr>
</tbody>
</table>

2.1.2 Types of Ultra-High Performance Concrete (UHPC)

There are different types of Ultra High Performance Concrete, which are based on the different curing methods.

2.1.2.1 Reactive Powder Concrete (RPC)

RPC is one such types of UHPC, which is cured through heat treatment process to accelerate the mechanical properties of concrete.

Pierre-Claude Aïtcin described the RPC as follows (5)

“We know how to make 150 MPA (21.8 ksi) concrete on an industrial basis. At such a level of strength the coarse aggregate in the concrete becomes the weakest link in the
concrete, the only option is to remove the coarse aggregates in order to increase the compressive strength and make a reactive powder concrete which has strength more than 200 Mpa (29 ksi). When the sand is replaced by metallic powder, the compressive strength increases to 800 Mpa (116 ksi).

Reactive powered concrete was developed by Bouygues laboratory in France 1990 by Richard and Cheyrezy. In reactive powdered concrete the pozzolanic reaction is accelerated by heat curing process. There exist two products which are RPC200, RPC800. RPC 200 (29 ksi) uses fine quartz, cement, silica fume to form the cementitious matrix and small steel fibers for enhancing the ductility. The fibers were 13 mm long and 0.15 mm in diameter. Addition of high range water reducers makes the concrete workable same as the conventional concrete. Richard and Cherezy (1994) initially tested the mix design of RPC200 which is cured at ambient temperature for 28 days and obtained the compressive strength of 170 MPA (24.5 ksi), and 230 MPA when cured at 90°C for 6-12 hours after pre-curing at ambient temperature for 2 days. RPC800 is restricted in its use for small or medium sized pre-fabricated structural elements. The composition of RPC 800 is quite similar to RPC200. The steel fibers are replaced by stainless steel fibers, less than 3 mm long. RPC 800 is cured at 250°C after demolding. Richard and Cherezy made use of steel powder instead of quartz sand and recorded the compressive strength of 800 MPA (116 ksi).

Richard and Cherezy (1994) have proposed two mix designs for reactive powder concrete. One is 200 MPA (29 ksi) concrete and other one is 800 MPA (116 ksi). The mix proportions are given below in Table 2 and Table 3
Table 2 - RPC 200 Mix Design (6)

<table>
<thead>
<tr>
<th>Material</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Portland cement – Type V</td>
<td>955 kg/m³</td>
</tr>
<tr>
<td>Fine sand (150-400 micron)</td>
<td>1051 kg/m³</td>
</tr>
<tr>
<td>Silica fume (18 m²/g)</td>
<td>229 kg/m³</td>
</tr>
<tr>
<td>Precipitated silica (35 m²/g)</td>
<td>10 kg/m³</td>
</tr>
<tr>
<td>Silica fume (18 m²/g)</td>
<td>229 kg/m³</td>
</tr>
<tr>
<td>Precipitated silica (35 m²/g)</td>
<td>10 kg/m³</td>
</tr>
<tr>
<td>Super plasticizer (Polycarboxylate)</td>
<td>13 kg/m³</td>
</tr>
<tr>
<td>Steel fiber</td>
<td>191 kg/m³</td>
</tr>
<tr>
<td>Total water</td>
<td>153 kg/m³</td>
</tr>
</tbody>
</table>

Table 3 - RPC 800 Mix Design (6)

<table>
<thead>
<tr>
<th>Material</th>
<th>Quantity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Portland cement – Type V</td>
<td>1000 kg/m³</td>
</tr>
<tr>
<td>Fine sand (150-400 micron)</td>
<td>500 kg/m³</td>
</tr>
<tr>
<td>Silica fume (18 m²/g)</td>
<td>230 kg/m³</td>
</tr>
<tr>
<td>Ground quartz (4 microns)</td>
<td>10 kg/m³</td>
</tr>
<tr>
<td>Super plasticizer (Polycarboxylate)</td>
<td>18 kg/m³</td>
</tr>
<tr>
<td>Steel fiber</td>
<td>630 kg/m³</td>
</tr>
<tr>
<td>Total water</td>
<td>180 kg/m³</td>
</tr>
</tbody>
</table>

Richard and cheyrezy 1995 recommended few steps to develop ultra-high performance concrete, which includes

- Removal of coarse aggregates to maintain the homogeneity
- Use of silica fume and other supplementary cementitious materials for pozzolanic reaction
- Application of presetting pressure for better compaction
- Post set heat treatment for enhancing the mechanical properties
- Enhancement of ductility by adding fibers
- Usage of super plasticizer to reduce water cement ratio (w/c) and improve workability
- Heat treatment during curing process can enhance the chemical reaction and help to gain strength

2.1.2.2 Thermal Treatment

Due to the very low water to cementitious material ratio in UHPC, the full hydration of cement and silica fume are never reached. Improved performance of UHPC has been observed after thermally treating UHPC using combinations of heat stream, pressure treatments. Thermal treatment allow continued hydration of Portland cement and pozzolanic reaction of the silica fume (Gatty et al. 1998; Cheyrezy et al. 1995). Graybeal (2006) observed improved durability characteristics including increase in resistance to chloride penetration and abrasion resistance. All these findings indicates that particle packing and different curing effects the characteristics of the UHPC. The primary function of heat treatment is to enhance the hydration reaction and further reduce the porosity and enhance other properties of concrete. Heat treatment can vary from 194° to 752° F (90 to 400° C) and heat treatment may last from 48 hours to 6 days. Typically UHPC is treated for 48 hours at 90°C.

Zanni et al (1996), found that the formation of hydrates at eight hours was 10% at heat treatment of 90°C compared to 55% heat treatment at 250°C, it means that the rate of hydration process increases with increase in curing temperature. The pozzolanic reaction of silica fume greatly depends on the temperature of curing, heat curing has the potential to accelerate the pozzolanic reaction (Zanni et al 1996; Richard and cheyrezy 1995). The increased pozzolanic reaction of silica fume reduces the porosity. Richard and
cheyrezy (1995) claim that the overall porosity will have no effect due to heat treatment but the secondary porosity is converted into small diameter porosity. (41) confirms this finding and states that sizes of the pores can be reduced to several smaller magnitude through heat treatment. Hydration reaction of untreated UHPC develops very quickly initially and drops down as all the mixed water is consumed.

2.1.2.3 Ductal Ultra High Performance Concrete (Patented Product)

In the early 1990’s two separate French contractors, Eiffage Group and Bouygues construction, with the help of Sika Corporation and Lafarge Corporation respectively, developed two different UHPC’S which exhibits similar properties. Eiffage Group with Sika Corporation created Beton Special Industrial (BSI) which is considered to be coarser (contains coarse aggregates) (43) compared to other UHPC’S, and the partnership between Bouygues construction and Lafarge Corporation produced Ductal (7).

The development of UHPC have benefited from improved aggregate gradation and use of high range water reducing agents or super plasticizers. UHPC was first developed as reactive powder concrete (RPC) with compressive strength ranging from 29 to 116 ksi. The high strengths were possible by completely eliminating coarse aggregates and improving microstructure by heat treatment (Richard and Cheyrezy) (6). Fibers really does not increase the strength, but they make the concrete ductile and improve tensile strength (7). Ductal is an ultra-high performance fiber reinforced concrete technology that offers superior compressive strength more than 29000 psi and flexural strength of 6000 psi. By utilizing the unique advantages of these properties, designers can create economical sections. Ductal provides the durability and impermeability against corrosion, abrasion and impact. The strength of ductal allows to design the section without
the passive reinforcement. Advantages of using Ductal would be reduced formwork, labor and maintenance costs on the other hand the disadvantage is high initial cost (8).

Properties of ductal. Figure 3, shows the plot of equivalent stress vs deflection of Lafarge Ductal

- Compressive strength Greater than 200 Mpa (>29,000 Psi)
- Direct tension up to 10 Mpa (1450 Psi)
- Flexural strength up to 40 MPA (6000 Psi)
- Abrasion resistance Similar to natural rock
- Impermeability Almost no carbonation or penetration of chlorides

Figure 3 - Equivalent stress vs deflection of Lafarge Ductal (9)
Table 4 and Table 5 shows typical values of strength and durability properties of ductal.

### Table 4 - Typical values of DUCTAL (9)

<table>
<thead>
<tr>
<th>Property</th>
<th>3 days (thermal treatment at 90°C for 48 hours)</th>
<th>28 days (wet curing)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Curing period</td>
<td>3 days</td>
<td>28 days</td>
</tr>
<tr>
<td>Compressive strength</td>
<td>235 MPA (34,075 Psi)</td>
<td>195 MPA (28,275 Psi)</td>
</tr>
<tr>
<td>Flexural strength</td>
<td>45 MPA (6,525 Psi)</td>
<td>40 MPA (5,800 Psi)</td>
</tr>
<tr>
<td>E- Modulus</td>
<td>60 GPA (8,702,264 Psi)</td>
<td>57 GPA (8,267,151 Psi)</td>
</tr>
</tbody>
</table>

### Table 5 - Durability properties of Ductal (9)

<table>
<thead>
<tr>
<th>Property</th>
<th>Index</th>
<th>HPC(High Performance concrete)</th>
<th>DUCTAL</th>
</tr>
</thead>
<tbody>
<tr>
<td>Abrasion</td>
<td>Volume loss</td>
<td>2.75</td>
<td>1.2</td>
</tr>
<tr>
<td>Freeze Thaw</td>
<td>%</td>
<td>90 %</td>
<td>100 %</td>
</tr>
<tr>
<td>Carbonation</td>
<td>Depth of penetration</td>
<td>2 mm</td>
<td>0</td>
</tr>
<tr>
<td>Chloride ion diffusion</td>
<td>X10^{-12} m^2/s</td>
<td>0.5</td>
<td>0.02</td>
</tr>
<tr>
<td>Post curing shrinkage</td>
<td>10^{-6}</td>
<td>300</td>
<td>0</td>
</tr>
</tbody>
</table>

2.1.2.3.1 Lafarge ductal composition

UHPC consist of materials similar to conventional concrete like Portland cement, silica fume, water, quartz sand. Coarse aggregates are eliminated in this composition. Additional to conventional concrete includes steel fibers (typically 0.008 in dia X 0.5 in long) and high range water reducing agents. The combination of these materials make a dense packing matrix which improves mechanical and rheological properties. Basic constituents of UHPC are shown Table 6.
Table 6- Mix proportions of Ductal Ultra High Performance Concrete (7)

<table>
<thead>
<tr>
<th>Constituent</th>
<th>Lb/yd³</th>
<th>% by weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Portland cement</td>
<td>1,180-1,1710</td>
<td>27-38</td>
</tr>
<tr>
<td>Silica fume</td>
<td>385-530</td>
<td>8-9</td>
</tr>
<tr>
<td>Ground quartz</td>
<td>0-390</td>
<td>0-8</td>
</tr>
<tr>
<td>Fine sand</td>
<td>1,293-1,770</td>
<td>39-41</td>
</tr>
<tr>
<td>Fibers</td>
<td>245-320</td>
<td>5-8</td>
</tr>
<tr>
<td>Super plasticizer</td>
<td>20-30</td>
<td>0.5-1.0</td>
</tr>
<tr>
<td>Water</td>
<td>260-350</td>
<td>5-8</td>
</tr>
<tr>
<td>Water/ cementitious ratio</td>
<td>0.14-0.27</td>
<td>----------</td>
</tr>
</tbody>
</table>

Both quartz sand and quartz powder contribute to the optimized packing. The most permeable portion of concrete is Interfacial transition zone (ITZ) between coarse aggregate and cement matrix (44), therefore elimination of coarse aggregates tends to improve the durability properties of UHPC. Reduction of ITZ zone increases the tensile strength and reduces the porosity (45). By reducing the water to make the concrete flowable, it is necessary to use poly carboxylate based super plasticizer which also help in improving the durability properties. Portland cement content in the UHPC is higher compared to normal strength concrete (NSC) and even compared to (HSC). The less water cement ratio prevents all the cement particles to hydrate. After thermal treatment the unhydrated particles exists in the matrix and acts as a particle packing material. Cements with low Blaine fineness, cements with high proportions of tricalcium aluminate (CA₃) and tricalcium silicates (C₃S) are desirable for the production of UHPC. The reason is CA₃, C₃S contribute to the high early strength and lower blaine fineness reducing the water demand. The addition of silica fume fulfils roles like particle packing and increase in the flowability due to spherical shape. Also the pozzolanic reaction leads to hydration of additional calcium silicate leading to additional strength.
2.1.3 Fiber Reinforced Concrete (FRC)

Concrete containing cement, water, fine aggregates, coarse aggregates, and discontinuous discrete fibers is called fiber reinforced concrete (FRC). The relationship between concrete and steel has been considered the major breakthrough in the construction industry. Concrete is strong in compression and weak in tension and brittle in nature. To compensate this shortcoming of concrete, steel is added so that it can provide the tensile strength. Traditional steel reinforcement is able to distribute the tensile strain forces that causes concrete to crack. This system has equal advantages and disadvantages (10). The problem with employing steel in concrete over time leads to corrosion due to ingress of chloride ions. In the north eastern states of United States, this is quite common problem where deicing salts are used which favors the formation of rust. Rust has a volume between 4 to 10 times the iron. The volume expansion produces larger tensile stresses in concrete which initiates cracks and results in spalling of concrete. Fibers promote to improve post peak ductility performance, pre crack tensile strength, fatigue strength, impact strength and eliminate temperature and shrinkage cracks (11). Steel is very expensive to purchase compared to concrete and the placement of steel takes time and labor cost. Most importantly steel is highly corrosive in nature which leads to steel cancer which means steel in the concrete corrodes and very expensive to repair and sometimes it leads to demolition of the structures (10).

Several different types of fibers have been used to reinforce the cement based matrices. The choice of fibers varies from synthetic organic fibers such as polypropylene, polyester, and polyethylene, synthetic inorganic such as steel or glass fibers, natural fibers such as wood, bamboo, elephant grass. Currently the commercial products are reinforced
with steel, glass, polyester and polypropylene fibers. The selection of the type of the fiber depends on properties such as diameter, length and the extent of the fibers affects properties of the cement matrix (12).

Even though the market for fiber reinforced concrete is very small when compared to overall production of the concrete, in North America there has been a yearly growth of 20% and throughout the world yearly consumption of fibers in concrete is 300,000 tons (13).

Classification of fiber volume fraction (13)

1) Low volume fraction (1%)

2) Moderate volume fraction (1 to 2%)

3) High volume fraction (more than 2%)

2.1.3.1 Low volume fraction

Low volume fraction fibers are used to reduce the shrinkage cracking. These fibers used in slabs and pavements which have large exposed area leading to high shrinkage crack. Less than 1% is generally considered as low volume fraction. (<1%). Fibers are distributed in all directions making an efficient load distribution.

2.1.3.2 Moderate volume fraction

Presence of fibers at this volume fraction increases the modulus of rupture, impact resistance and fracture toughness. They are used in the structures where energy absorption is required, improved capacity against delamination and fatigue.

2.1.3.3 High volume fraction

The fibers used at this fraction lead to strain hardening of the composites. Because of this improved behavior, it is often refereed as high performance fiber reinforced
composites. Ultra-High Performance Fiber Reinforced Concretes (UHPFRC) are often developed using high volume fraction fibers.

2.1.3.4 Durability

When the fiber reinforced concrete is well compacted and cured concrete containing steel fibers possess excellent durability properties as long as the fibers protected by cement paste. Long term durability test on concrete which contains steel fibers was conducted in Battelle laboratories Columbus showed minimum corrosion of fibers and no adverse effect after 7 years of exposure to deicing salts (13). Figure 4 shows Crack opening vs tensile stress of Fiber reinforced concrete.

- Improves the toughness of the concrete
- Flexural strength is increased by 30%
- Improves tensile strength of concrete

![Figure 4 - Crack opening vs tensile stress of fiber reinforced concrete (13)](image-url)
2.1.4 Self-consolidating concrete (SCC)

In the last 10 years there has been significant improvement in the new concretes which today are providing innovative solutions, with regard to design, placement, and aesthetic view. Concrete has changed enormously in the 20th century due to development in mineral and organic chemistry and improvement in production techniques. Modern concrete is more than just cement water and aggregates. Modern concrete contains mineral components, chemical admixtures and fibers and many more. The development of smart concretes results from the development of new science of concrete, new science of admixtures and use of sophisticated machines, new apparatus to observe the micro and nano structures (14). Research carried out to improve workability of concrete without affecting its strength is called as self levelling or self-compacting concrete which is extremely fluid, stable and homogenous. Self-consolidating concrete is an advanced type of concrete that can flow and consolidate under its own mass without any vibration which can pass through intricate geometrical configurations and can resist segregation. It has been used successfully in precast and ready mixed concrete applications (15).

SCC is defined in terms of its workability. The main abilities of SCC are its ability to flow under its own mass (filling ability), its ability to pass through congested (crowded) reinforcement (passing ability) and its ability to resist segregation (segregation resistance). American Concrete Institute (ACI) defines SCC as “highly flowable, non-segregating concrete that can spread into place, fill the formwork and encapsulate the reinforcement without any mechanical vibration. The Precast/Prestressed Concrete Institute defines SCC as “a highly workable concrete that can flow through densely reinforced or geometrically complex structural elements under its own weight and fills the voids without any
segregation or excessive bleeding (15). It is generally accepted that SCC was first developed in Japan in 1980s in response to the lack of skilled labor and improve the durability. Collepardi (2003) (47) states that self levelling concretes were studied as early as in 1975 and used in commercial applications in Europe, and the United States Asia in the 1980s. The use of SCC has gradually increased throughout the world since 1980s and gained the momentum in the late 1990s. One of the first high profile application of SCC was the Akashi Kaikyo Bridge in Japan (46).

The advantages of SCC (15) includes

- Improved surface finish and reduced need to repair defects such as bug holes and honeycombing
- Reduced construction costs due to reduced labored costs and reduced equipment purchase and maintenance cost
- Increased construction productivity, improved jobsite safety, and enhanced concrete quality.
- Faster unloading of ready mixed concrete trucks
- Improved concrete strength and durability
- Improved working conditions with fewer accidents due to elimination of mechanical vibrators

The disadvantages of SCC includes

- Increased material costs especially for admixtures and cementitious materials
- Increased technical expertise require to develop and control mixtures
- Delayed setting times in some cases due to the use of admixtures
2.1.5 Ultra High Performance Fiber Reinforced Concrete (UHPFRC)

Ultra-High Performance Fiber Reinforced Concrete (UHPFRC) is a material that reaches higher compressive strength at shorter time, and considerable tensile strength associated to higher strains. Good mechanical properties of UHPFRC are possible because of lower W/C ratio, higher steel fiber content, and dense matrix, as a result high ductility is achieved. These characteristics and high control requirements make this material suitable for the design of precast prestressed elements. The freedom for design shapes (due to removal of most of the conventional steel) and potential elements slenderness are also interesting to design more elegant structures (16). The fibers in the UHPC provides tensile capacity across the cracks, resulting in high shear capacity in bending members. Typically additional reinforcement for shear is not required as per Graybeal 2006.

2.1.5.1 Advantages of UHPFRC

Fibers added to concrete improves its mechanical resistance and ductility, thereby reducing the plastic shrinkage, improving resistance to abrasion, fire and impact. With such kind of materials engineers able to design structures to resist severe conditions. One of the most important properties of steel fiber reinforced concrete (SFRC) is its superior resistance to cracking and crack propagation (17). As a result of this ability to arrest cracks, fiber composites possess increased extensibility and tensile strength, both at first crack and at ultimate under flexural loading. The fibers are able to hold the matrix even after the extensive cracking. The net result of this is post-cracking ductility which is unheard in the normal conventional concrete. The transformation from brittle to ductile type of concrete would increase substantially the energy absorption characteristics of the fiber composite and its ability to withstand repeatedly applied shocks, or impact loading (Johnston, 2001).
Over the years considerable efforts to improve the behavior of the cementitious materials by incorporating fibers has led to the invention of UHPFRC. This novel material provide the designers with unique combination of

1) Extremely low permeability: which prevents the ingress of chlorides, which is the main reason for the corrosion of the reinforcing bars (19)

2) UHPFRC or SHCC exhibits a tensile strain hardening response associated to the formation of finely distributed microcracks. UHPFRC with suitable fibrous mixes have a dense matrix, a very low capillary absorption of liquids and limited shrinkage. Compressive strength more than 150 MPA (21,750 Psi) and up to 20 MPA (2900 psi) direct tension with considerable strain hardening and softening behavior (20)

3) Higher compressive strength, tensile strength, low porosity, improved microstructure, homogeneity due to removal of bigger size gravel, high flexibility with addition of fibers. As a result of superior performance UHPFRC has found its application in the structures like storage of nuclear waste, bridges, roofs, piers, seismic resistant structures (21)

4) Durability issues of traditional concrete have been acknowledged for many years and significant funds are required to repair the existing infrastructures. UHPFRC possess good durability properties and require very less maintenance costs in its service life compared to traditional concrete (21)

5) UHPFRC enables structural members to be built without the need of transverse reinforcement because of its advantageous flexural strength (18)
Since UHPC can lead to longer span structures with reduced member sizes compared to normal or high strength concrete, a significant reduction in volume and self-weight would be expected with UHPC members. Figure 5 below shows the UHPC, Steel, and Prestressed, reinforced concrete beams with equal moment capacities, the UHPC beam required only half of the section depth when compared to reinforce or prestressed concrete beams which in turn reduced the weight by 70%. The UHPC beam has the same section depth as steel beam. The cross sectional area of UHPC compressive members may be reduced compared to normal concrete members due to UHPC’s high compressive strength (22). Table 7 describes Difference between UHPC and different reinforced concrete.

Table 7 – Difference between UHPC and different reinforced concrete (22)

<table>
<thead>
<tr>
<th>Material</th>
<th>UHPC</th>
<th>Steel</th>
<th>Prestressed Concrete</th>
<th>Reinforced Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth</td>
<td>14 in (360 mm)</td>
<td>14 in (360 mm)</td>
<td>28 in (700 mm)</td>
<td>28 in (700 mm)</td>
</tr>
<tr>
<td>Weight</td>
<td>94 lb/ft (141 kg/m)</td>
<td>75 lb/ft (110 kg/m)</td>
<td>313lb/ft (466 kg/m)</td>
<td>355lb/ft (528 kg/m)</td>
</tr>
</tbody>
</table>

Figure 5 – Difference showing between UHPC Steel Reinforced Concrete and PSC (22)
2.1.6 Large Scale Applications

There are many structures around the world constructed using Ultra High Performance concrete.

2.1.6.1 Sherbrooke Pedestrian Bridge:

Development of UHPC began in 1990’s Sherbrooke pedestrian bridge located in Quebec, Canada is the first Bridge in the world which is made of Ductal Ultra High performance concrete in 1997. Bridge spanning 60m across Magog River with a space truss. The main span is an assembly of six 10 m prefabricated match – cast segments. The bridge deck is of thickness 30 mm (semioli 2001) with prestressed with transversal ribs every 1.25 m. It is bedded on 2 longitudinal beams to which diagonals are connected. The truss webs are made of RPC confined in stainless steel tubes. This bridge made of Ductal product superior and ultra-high performance and opened up the market for innovative materials. This project won a precast prestressed concrete institute design award (23). To develop an understanding on how UHPC works in practical applications, a long term monitoring program was implemented on this bridge to monitor deflections and forces. In 1997, UHPC’s durability received a test when it was to replace steel beams in the cooling towers of the Cattenom power plant in France. The environment was extremely corrosive and UHPC was chosen with an expectation of reducing the maintenance cost. Three years later AFGC-SETRA working group visited the site and observed that under normal layer of sediment, no deterioration of UHPC (Resplendino and Petitjean 2003) (7). Figure 6 describes the cross section of Sherbrooke pedestrian bridge and Table 8 shows the composition of RPC used in the construction. Figure 7 shows the general view of Sherbrooke pedestrian bridge.
Figure 6- Cross section of Sherbrooke pedestrian bridge (24)

Table 8- Composition of RPC used in the construction of Sherbrooke Bridge (24)

<table>
<thead>
<tr>
<th>Component</th>
<th>Amount (kg/m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ASTM type 2 cement</td>
<td>710</td>
</tr>
<tr>
<td>Silica fume</td>
<td>230</td>
</tr>
<tr>
<td>Ground quartz</td>
<td>210</td>
</tr>
<tr>
<td>Silica sand</td>
<td>1010</td>
</tr>
<tr>
<td>Steel fibers</td>
<td>190</td>
</tr>
<tr>
<td>Super plasticizer</td>
<td>19</td>
</tr>
<tr>
<td>Water</td>
<td>200</td>
</tr>
</tbody>
</table>

Figure 7 - Sherbrooke pedestrian bridge (22)
2.1.6.2 Wapello Road Bridge:

In 2003 Wapello county and Iowa Department of transportation used UHPC in prestressed concrete beams in a bridge replacement project. The beams are pre tensioned using 15.4 mm diameter low relaxation strands. No reinforcing steel except to provide to provide composite action between beam and deck was used. The replacement bridge is 34 m span bridge with 3 beam cross sections and total width of 8.3 m, 203mm cast in place deck. Beam spacing is 2.9 m with approximately 1.2 m overhanging. A total of 50 m³ UHPC based Ductal premix was used to prestressed beams as given in Table 9. Figure 8 shows the general view of the bridge. Figure 9 shows the cross section of the bridge.

Table 9 - The composition of UHPC used in Wapello Bridge (24)

<table>
<thead>
<tr>
<th>Component</th>
<th>Amount (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ductal Premix</td>
<td>2194</td>
</tr>
<tr>
<td>Metallic fiber</td>
<td>156</td>
</tr>
<tr>
<td>Super plasticizer</td>
<td>30</td>
</tr>
<tr>
<td>Water</td>
<td>131</td>
</tr>
</tbody>
</table>

Figure 8 - General view of Wapello Road Bridge (24)
2.1.6.3 Glenmore trail at Legsby Road

The Glenmore trail pedestrian bridge is a unique structure over a vital arterial road in southwest Calgary. The challenges experienced were, the limited space for the construction of piers, and the busy roadway that could not be blocked for a longer period of time. These concerns led to the construction of cantilever type Bridge which supports a UHPC precast drop in girder. It is a single span 53 m bridge that stretches across 8 lanes of traffic. It consist of two cantilevered high performance concrete abutments and a drop in “T section” UHPFRC ( ductal) girder with an arch. The girder is 33.6 m long, 1.1 m deep at the mid-span with a 3.6 m wide deck and weight is approximately 100 tons. It is constructed with 13 mm steel fibers post tensioned with 15 mm strands. Glass fiber reinforced plastic (GFRP) bars were utilized as passive reinforcing system. The girder required 40 m³ of material resulting in the largest single monolithic pour of ductal in the
world to date. To ensure proper efficient mixing high shear mixers were used. Figure 10 shows the general view of Glenmore trail at Legsby Road.

![Glenmore trail at Legsby Road](image)

Figure 10- Glenmore trail at Legsby Road (24)

2.1.6.1.4 Rainy Lake – CN rail overhead bridge

The original CN overhead bridge was constructed in 1962 on Ontario highway, near Rainy Lake Ontario, Canada. The existing bridge deck had reached its useful life and was in need of reconstruction. An innovative way was developed to reconstruct the bridge deck. The existing deck was removed transversely, one half at a time, while maintaining full traffic volume and replaced with a new precast deck panel system. GFRP bars were used as a top reinforcement to avoid the problem of the corrosion due to deicing salts. A challenge faced by highway authorities was durability of the joints due to constant flexing truckloads and corrosion of the rebar crossing the joints due to salts. To minimize the corrosion potential, non-corrosive reinforcing bar was used at the top mat and joint size was minimized to provide the least possible shrinkage across the joint. The Ductal joint fill
material has less porosity, improved flexural strength, improved toughness, superior freeze/thaw resistance. The self levelling material was then covered with a plywood to protect against moisture loss. After 4 days of field curing the UHPFRC material was ground smooth in the area of any high spots. Traffic was opened onto the reconstructed deck and the same process was done for the other decks. Figure 11 shows the joints of CNO overhead bridge.

![Figure 11- Joints of CNO overhead bridge (18)](image)

2.1.6.1.5 The Seoul and Sakata Mirai Footbridges

Sakata Mirai footbridge is planned to replace the old prestressed concrete pedestrian bridge that was built 40 years ago. In 2002 a 50 m span footbridge has been constructed using ductal for the very first time in Japan. In order to take the full advantages of the characteristics of UHPFRC and especially to use it without passive reinforcement the structural concept and unique points of Sakata Mirai Bridge are as follows

1) Ultimate thin slab (50mm) and web (80 mm) were employed to reduce the dead weight

2) Perforated webs were used for the sake of design view and reduce the dead weight
3) Eight precast segments were transversely and longitudinally connected to each other by wet joint.

Table 10 shows the composition of Ductal used in the construction

<table>
<thead>
<tr>
<th>Unit</th>
<th>Mixed ingredients</th>
<th>Water</th>
<th>fibers</th>
<th>Super plasticizer</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kg/m³</td>
<td>2254</td>
<td>180</td>
<td>157</td>
<td>24</td>
</tr>
</tbody>
</table>

2.1.6.1.5.1 Material performance applied for footbridge

By reducing the water cement ratio, defects such as microcracks and pore spaces can be minimized. Enhancement of compacted density was achieved by optimization of granular mixture, enhancement of microstructure was achieved by heat treatment and finally enhancement of ductility was achieved by addition of steel fibers. Flow diameter of 240-260 mm was achieved even after including 2% fibers by volume. The sinking or segregation of the fibers does not occur because of the viscosity of the paste. Figure 12 shows the general view of Sakata Mirai Bridge.

Figure 12- Sakata Mirai Bridge (25)
2.1.6.1.6 The First Road Bridge Made Of Uhpc– Bourg Les Valence Bridges (21)

During the years 2000 and 2001, the French Government, with the assistance of the Service d’Etudes Techniques des Routes et AutoRoute (SETRA) and the Centre d’Etudes Techniques de l’Equipement (CETE) of Lyon, built the world’s first UHPC bridge. Each bridge has two isostatic spans of 65 ft. (20 m). The road deck was made continuous by placing in situ UHPC between two spans. Transversely, both the decks are identical, they were made from an assembly of fiver pi shaped precast beams made of BSI. Figure 13 shows the longitudinal cross section of the bridge.

![Figure 13- Longitudinal cross section (21)](image)

All the beams were prestressed by pre tension. There is no transverse prestress, and no transverse passive reinforcement, except where pi shaped beams are transversely jointed together. Figure 14 explains the typical cross section of the bridge.

![Figure 14- Typical cross section of Bourg les valence bridges (21)](image)
This road bridge required to settle special calculation methods and design rules which are not currently covered by codes for the type of the concrete employed.

2.1.6.1.7 Other Applications

The roof of a clinker silo in Joppa Illinois was constructed with UHPC in 2001. The 24 wedge shaped precast panels with thickness of 0.5 in covered the 58 ft. diameter silo. By Utilizing UHPC labor and time was minimized. In Detroit Michigan, the cement industry uses UHPC to make the columns with smaller sections in the cement terminal, which allowed five feet more width truck clearance for loading (7). Figure 15 shows the Joppa Clinker silo, Figure 16 shows the 54’ height UHPC column.

![Figure 15 - UHPC panels on silo (7)](image1)

![Figure 16 - 54 feet UHPC column (7)](image2)

Apart from these main civil engineering structures mentioned above, some other applications have been constructed using UHPC.

- The construction of architectural wall panels for Rhodia head office in Aubervilliers
- The realization of punched and thin acoustic sound panels for the underground Monaco railway station 1500 m$^2$ panels.
- At the beginning of 2003, 30 m$^3$ poured in steel tubes were casted for making the columns of Keen Sofia Museum Madrid (Spain).
- The construction of 6300 anchor plates (Figure 17) with polymer fibers and 200 plates with steel fibers for reinforced, earth located at the sea front on La Reunion Island.

![Image of Ductal anchor plates](image1)

Figure 17- Ductal anchor plates (7)

- Injection of curved saddles to keep the cables in the pylons of Sungai Muar Bridge in Malaysia, as shown in the Figure 18.

![Images of Sungai Muar cable stayed bridge](image2)

Figure 18 - Prefabricated saddles used for Sungai Muar cable stayed bridge in Malaysia (7)
2.1.7 Materials of UHPFRC

Materials used for UHPFRC are little different to what will be used in traditional concrete. UHPFRC contains no large aggregates. The largest granular material used is sand ranging from 150 to 600 microns along with high amount of cement and silica fume having diameter small enough to fill the interstitial spaces between the cement particles. Dimensionally the largest constituent in the mix is the steel fiber which has the diameter of 0.008” and 0.5” length. Steel fibers are able to reinforce the concrete matrix on micro level. To improve and ensure the self levelling, high workability is required and achieved by using different HRWRA’s.

2.1.7.1 Portland Cement

Portland cement is the foremost among the construction materials which acts as binder that holds fine and micro fine particles. In the simple form, concrete is a mixture of paste and aggregates, the paste consist of Portland cement and water, and coats the surface of fine and coarse aggregates. Its versatility, and adaptability, as evidenced by the many types of construction in which it is used and minimum maintenance required during its service life.

Chemical composition of cement:

- Lime or calcium oxide, $\text{CaO}$: from limestone, chalk, shale
- Silica $\text{SiO}_2$: from sand, old bottles, clay or argillaceous rock
- Alumina, $\text{Al}_2\text{O}_3$: from bauxite, recycled aluminum, clay
- Iron, $\text{Fe}_2\text{O}_3$: from clay. Iron ore, fly ash
- Gypsum $\text{CaSO}_4\cdot2\text{H}_2\text{O}$: found together with limestone
Table 11 explains the chemical shorthand of the compounds and Table 12 explains the chemical composition of clinker.

**Table 11- Chemical shorthand (48)**

<table>
<thead>
<tr>
<th>Compound</th>
<th>Formula</th>
<th>Shorthand</th>
</tr>
</thead>
<tbody>
<tr>
<td>Calcium oxide (lime)</td>
<td>CaO</td>
<td>C</td>
</tr>
<tr>
<td>Silicon dioxide (silica)</td>
<td>SiO₂</td>
<td>S</td>
</tr>
<tr>
<td>Aluminum oxide (alumina)</td>
<td>Al₂O₃</td>
<td>A</td>
</tr>
<tr>
<td>Iron oxide</td>
<td>Fe₂O₃</td>
<td>F</td>
</tr>
<tr>
<td>Water</td>
<td>H₂O</td>
<td>H</td>
</tr>
<tr>
<td>Sulfate</td>
<td>SO₃</td>
<td>S</td>
</tr>
</tbody>
</table>

**Table 12- Chemical composition of clinker (48)**

<table>
<thead>
<tr>
<th>Compound</th>
<th>Formula</th>
<th>Shorthand form</th>
<th>% by weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tricalcium aluminate</td>
<td>Ca₃Al₂O₆</td>
<td>C₃A</td>
<td>10</td>
</tr>
<tr>
<td>Tetracalcium aluminoferrite</td>
<td>Ca₄Al₂Fe₂O₁₀</td>
<td>C₄AF</td>
<td>8</td>
</tr>
<tr>
<td>Belite or dicalcium silicate</td>
<td>Ca₂SiO₅</td>
<td>C₂S</td>
<td>20</td>
</tr>
<tr>
<td>Alite or tricalcium silicate</td>
<td>Ca₃SiO₄</td>
<td>C₃S</td>
<td>55</td>
</tr>
<tr>
<td>Sodium oxide</td>
<td>Na₂O</td>
<td>N</td>
<td>Up to 2</td>
</tr>
<tr>
<td>Potassium oxide</td>
<td>K₂O</td>
<td>K</td>
<td>Up to 2</td>
</tr>
<tr>
<td>Gypsum</td>
<td>CaSO₄.2H₂O</td>
<td>CSH₂</td>
<td>5</td>
</tr>
</tbody>
</table>
Properties of cement compounds

- Tricalcium aluminate, \( C_3A \):
  
  It liberates a lot of heat during the early stages of hydration, but has little strength contribution. Gypsum slows down the hydration of \( C_3A \). Cement low in \( C_3A \) is sulfate resistant.

- Tricalcium silicate, \( C_3S \):
  
  This product is largely responsible for initial set and high early strength.

- Dicalcium silicate \( C_2S \):
  
  \( C_2S \) hydrates and hardens slowly. It is largely responsible for the gain after one week.

- Ferrite, \( C_4AF \):
  
  This product is a fluxing agent which reduces the melting temperature of the raw material in the kiln. It hydrates rapidly, but doesn’t not contribute much to the strength.

Chemical reactions during hydration:

When water is added to cement the following reactions occur

- Tricalcium aluminate reacts with gypsum in the presence of water and forms ettringite and heat. Ettringite does not contribute to the strength.
  
  \[
  \text{Tricalcium aluminate + gypsum + water } \rightarrow \text{ettringite + heat}
  \]
  
  \[
  C_3A + 3\text{CSH}_2 + 26H \rightarrow C_6\text{AS}_3\text{H}_32, \Delta H = 207 \text{ cal/g.} \quad (1)
  \]

- Tricalcium silicate (alite) is hydrated to produce calcium silicate hydrate, lime and heat
  
  \[
  \text{Tricalcium silicate + water } \rightarrow \text{calcium silicate hydrate + lime + heat}
  \]
  
  \[
  2C_3S + 6H \rightarrow C_3\text{S}_2\text{H}_3 + 3\text{CH}, \Delta H = 120 \text{ cal/g} \quad (2)
  \]
CSH has a short network fiber which greatly contributes to the initial strength of the cement glue

- Once all the gypsum is used up as per reaction (i), ettringite becomes unstable and reacts with remaining tricalcium aluminate to form monosulfate aluminate hydrate crystals

\[
\text{Tricalcium aluminate + ettringite + water} \rightarrow \text{monosulfate aluminate hydrate}
\]

\[
2\text{C}_3\text{A} + \text{C}_6\text{AS}_3\text{H}_{32} + 22\text{H} \rightarrow 3\text{C}_4\text{ASH}_{18} \quad \text{(3)}
\]

The monosulfate crystals are only stable in sulfate deficient solution. In the presence of sulfates, the crystals resort back into ettringite, whose crystal size is two and half times bigger than monosulfate. It is this increase in size which causes the cracks in the concrete when it is subjected to sulfate attacks.

Dicalcium silicate (belite) hydrates to form calcium silicate hydrate and heat

Dicalcium silicates + water → calcium silicate hydrate + lime

\[
\text{C}_2\text{S} + 4\text{H} \rightarrow \text{C}_3\text{S}_2\text{H}_3 + \text{CH} + \Delta \text{H} = 62 \text{ cal/g} \quad \text{(4)}
\]

Calcium silicate hydrate contributes to the strength of the cement paste. This reaction generates less heat compared to reaction (ii), it means, the contribution of CSH to the strength of cement paste would be slow initially. However this compound is responsible for the long term strength of the Portland cement concrete.

C-S-H or C_3S_2H_8 is called calcium silicate hydrate and it is the principle hydration product

The hardened cement paste (49)

Hardened cement paste consist of following,

Ettringite - 15 to 20%

Calcium silicate hydrate - 50 to 60%
Calcium hydroxide (lime) – 20 to 25%

Voids - 5 to 6% (in the form of capillary voids entrapped and entrained air)

2.1.7.2 Silica fume

Pozzolanic materials are very important in the production of UHPC. The most widely used pozzolanic material is silica fume. The terms micro silica, condensed silica, silica fume are often used to describe the byproducts extracted from the exhaust gases of ferrosilicon, silicon and other metal alloy smelting furnace. Mineral additions which are also known as mineral admixtures have been used in the concrete for many years. There are two types of admixtures that are commonly blended with cement these days. They are crystalline called as, hydraulically inactive and pozzolanic which are hydraulically active. Silica fume is very reactive pozzolona, which has very small particle size and larger surface area and high amount of SiO₂ content. Although silica fume is relatively old additive, however the amount as supplementary materials in the concrete mixture is not fully understood till now. Jianxin Ma Developed Ultra-High Performance Concrete and founded that optimal amount of silica fume should be more than 25% (by weight of cement) to get densest granular mixture. A. A. Elsayed in his experiment found that higher amounts of silica fume should be avoided because mixtures slump intend to decrease, but resistance to water penetration intend to increase. Kennouche S researched self-compacting concrete and founded that the best workability results are achieved when the silica fume is used 15% by weight of cement (37).
2.1.7.2.1 Undensified silica fume

Bulk density of undensified silica fume ranges from 200-350 kg/m$^3$. Due to low bulk density undensified silica fume is impractical to use in regular concrete works. It can be used in grouts, mortars, and in repair works (38).

2.1.7.2.2 Densified silica fume

Densified silica fume has the bulk density ranging from 500-650 kg/m$^3$ (40). In the densification process, ultra-fine particles becomes loosely agglomerated which makes the particles size bigger. Hence powder becomes easier to use when compared to undensified silica fume which is in the dust form. Densified silica fume is used in those places that used high shear mixer facilities such as concrete roof tile work, pre cast work, and ready mix power plants. The appearance of fine powder from black to slightly off white. There is a strong relationship between color and carbon content. The correlation is not straightforward though, since carbon may be present as a coke residue or silicon carbide which is additional to carbon is the influencing thing in imparting the color to the silica fume. Figure 19 shows the scanning electron microscopy of silica fume.

Figure 19- Scanning electron microscopy of condensed silica fume (40)
2.1.7.2.3 The Pozzolanic Reactions

In the presence of hydrating Portland cement, silica fume reacts with calcium hydroxide (CH) to form calcium silicate hydrate which mainly responsible for the strength of matrix (18)

(C₃S+C₂S) + H₂O → C-S-H + CH + S → C-S-H .......... (5)

Cement water silica

Mechanism by which silica fume modifies cement paste

Cohen, Olek and Dolch (1990) (50) have calculated that for a 15% silica fume replacement of cement, there are 2,000,000 particles of silica fume for each grain of cement in the concrete mixes. Therefore there is no surprise that silica fume has pronounced effects on concrete properties.

The strength of the transition zone between the coarse aggregates and the cement paste is less than that of bulk cement paste. Generally transition zone contains lot of voids because of the water underneath the aggregate particles. Calcium hydroxide (CH) forms more in this region than any other region. Without the presence of silica fume the CH particles grow larger and tend to orient parallel to aggregate particle surface (Monteiro, Maso and Olliver 1985). CH is weaker than calcium silicate hydrate (C-S-H), and when crystals are larger and strongly oriented parallel to aggregate surface they are easily separated (39).

As per report by ACI committee 234 on Guide for the Use of Silica Fume in Concrete following research work has been reported.

Mindess (1988) research says that presence of silica fume increases the strength of the concrete because of the increase in bond strength between cement pastes aggregate.
Wang et al (1986) findings says that a small addition of silica fume between 2 to 5% produced a denser structure in transition zone with consequent increase in fracture toughness and micro hardness.

Detwiler (1990) found that presence of silica fume also increased the fracture toughness of the transition zone between cement paste and steel.

Sellevold (1987) pointed out that the increased cohesion will benefit the hardened concrete in terms reduced segregation and bleed water pockets under coarse aggregates and reinforcing bars.

Monteiro and Mehta (1986) stated that silica fume reduces the thickness of the transition zone between aggregate particles and cement paste. This is one reason for reduction in bleeding.

In hardened concrete, silica fume particles increase the packing of the solid materials by filling the spaces between the cement grains in much the same way as cement fills the spaces between fine aggregates and fine aggregates fills the spaces between the coarse aggregate particles in concrete. Bache (1981) explained the theory of packing of solid particles and its effects on the material properties. Concrete is a composite material so its properties are not only effected by packing particle of the cement paste, but also their packing near to the aggregates surface. The figure shows how the minute amount of silica fume particles improve packing in the boundary zone. Since it is the weakest part in the concrete, it is very important packing in this region. Bache also stated that presence of silica fume in the concrete could reduce the water demand as the fume particles are occupying the spaces between the cement grains otherwise water would have occupied the
space between cement grains. This reduction in water applies to those systems with enough admixtures to reduce the surface forces.

Sellevold and Radjy (1983) reported on decrease in water demand for silica fume mixtures and stated that water reducing admixtures have greater influence on silica fume concretes. However in most of the concretes works with silica fume would increase the water demand due to larger surface area of the silica fume and would require the use of high range water reducing agents.

Ono, Asaga, and Diamon (1985) studied the cement silica fume system in lower water cement ratio systems (0.23) at 20°C. The amounts of CH present after the hydration of cement: silica fume ratios are 100:0, 90:10, 80:20, and 60:40 are given in the Figure 20.

At the high levels of silica fume all the CH is consumed in 28 days. At the lower levels i.e., at 10%, CH is almost reduced by 50% in 28 days. For the mixtures which contains 50% and more no CH is detected in 14 days. Hooton (1986) found that with 20% by volume replacement of silica fume, no CH was detected in 90 days at 23°C normal curing, while 10% silica fume reduced CH to 50% in 90 days at normal curing process.

![Figure 20](image)

Figure 20- Amount of calcium hydroxide in cement paste containing different amounts of silica fume (39)

Table 13 explains the effect of silica fume on mechanical and durability properties.
Table 13 - Silica fume and it’s effects on concrete properties (40)

<table>
<thead>
<tr>
<th>Concrete properties</th>
<th>Increase</th>
<th>Decrease</th>
<th>Enhancement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strength</td>
<td>x</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tensile strength</td>
<td>x</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Modulus of elasticity</td>
<td>x</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Flexural strength</td>
<td>x</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tensile ductility</td>
<td>x</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Air void content</td>
<td>x</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Freeze thaw durability</td>
<td></td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Abrasion resistance</td>
<td></td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Bond strength with steel fibers</td>
<td></td>
<td></td>
<td>x</td>
</tr>
<tr>
<td>Chemical attack resistance</td>
<td></td>
<td></td>
<td>x</td>
</tr>
<tr>
<td>Corrosion resistance of reinforcement steel</td>
<td></td>
<td></td>
<td>x</td>
</tr>
<tr>
<td>Alkali silica reactivity</td>
<td></td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Shrinkage</td>
<td></td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Permeability</td>
<td></td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Creep rate</td>
<td></td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Coefficient of thermal expansion</td>
<td></td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Density</td>
<td></td>
<td></td>
<td>x</td>
</tr>
<tr>
<td>Workability</td>
<td></td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Bleeding</td>
<td></td>
<td>x</td>
<td></td>
</tr>
</tbody>
</table>

2.2 EXPERIMENTAL INVESTIGATION

This section describes about the experimental findings of the various researchers all over the world.

2.2.1 Compressive Strength

In North America, cylinder compressive strength is widely used. In modern structural concrete, compressive strength is one of the most, in fact it is the most important property of concrete in terms of verifying the acceptability of wide range of concretes structures performance. Accurately and reliably verifying the compressive strength of the
Ultra-High performance concrete is a challenging task. Two standard methods of determining the compressive strength of the concrete are testing cylinders and cubes. National codes and specifications in North America, France, Japan, Australia and New Zealand define cylinders as the standard specimen, whereas much of the Europe and other reminders depend on cube compressive strength (26).

Accurate determination of compressive strength of High performance concrete is a difficult task these days due to preparation of end surface of the cylinders. An experimental program was conducted by Benjamin Graybeal and Marshall Davis to determine whether an alternate specimen types can be reliable for the testing of UHPFRC whose compressive strength ranges from 80 Mpa (11600 psi) to 200 MPA (29000 psi). 3”(75 mm), 4” (100mm) cylinders and 2” (50 mm), 2.78” (70.7 mm), 4”(100 mm) cubes are also been used for testing compressive strength. Cubes are preferred, where cylinder end preparation and machine capacity is a concern. Graybeal and Marshall Davis research focused on determining the viability of using reduced dimension cube specimen for the measurement of the compressive strength. Cylinders were tested according to ASTM C39 and the loading rate was modified to speed up the testing process. 150 psi/sec was adopted as the loading rate instead of original value of ASTM 35±7 psi/s.

A number of observations were made from the results. Firstly 70 mm and 50 mm size cubes tend to show similar strengths with all their confidence intervals overlapping, secondly 100 mm and 75 mm (3” and 4”) cylinders also tend to show similar results. Finally 50 mm (2”) cylinders tend to show similar or lower strengths as compared with all the other specimen types.
Table 14 gives the coefficient of conversions of compressive strength results. Figure 21 shows the compressive strength variation of different sized cylinder and cubes.

Table 14- Coefficients for conversion of compressive strength results (26)

<table>
<thead>
<tr>
<th>Specimen size</th>
<th>76 mm (3””) dia cylinder</th>
<th>102 mm (4””) dia cylinder</th>
</tr>
</thead>
<tbody>
<tr>
<td>100 mm cube (4” cube)</td>
<td>Multiply by 1</td>
<td>Multiply by 1</td>
</tr>
<tr>
<td>70.7 mm cube (2.78” cube)</td>
<td>Multiply by 0.94</td>
<td>Multiply by 0.93</td>
</tr>
<tr>
<td>51 mm cube (2” cube)</td>
<td>Multiply by 0.96</td>
<td>Multiply by 0.96</td>
</tr>
<tr>
<td>102 mm cylinder (4” cylinder)</td>
<td>Multiply by 1.01</td>
<td>---</td>
</tr>
<tr>
<td>76 mm cylinder (3” cylinder)</td>
<td>---</td>
<td>Multiply by 0.99</td>
</tr>
<tr>
<td>51 mm cylinder (2” cylinder)</td>
<td>Multiply by 1.08</td>
<td>Multiply by 1.07</td>
</tr>
</tbody>
</table>

Figure 21 – Compressive strength comparison (26)

Conclusions

1) 4” diameter cylinder, 3” diameter cylinder and 100 mm cubes (4”) are acceptable and interchangeable specimens for determination of compressive strength of UHPFRC.
2) The 70 mm (2.78”) cubes is an acceptable alternative specimen type for determination of strength of UHPFRC, in situations where end preparation and testing machine capacity are limited.

3) 51 mm cylinders exhibit significantly increased coefficient of variation.

2.2.2 Research carried out by Clemson University

Clemson University in South Carolina (27) have developed the high strength/ high performance grout material for application in shear keys in precast bridges. Four design mixes were prepared, in which two design mixes do not contain steel fibers and other two contains steel fibers. Table 15 explains all about the design mixes developed at the Clemson University.

Table 15- Quantity of materials per one cubic meter of UHPC mixtures (27)

<table>
<thead>
<tr>
<th>UHPC ID</th>
<th>UHPC Description</th>
<th>Quantity of materials of UHPC mixtures, kg</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Cement</td>
<td>Sand</td>
</tr>
<tr>
<td>UHPC 1</td>
<td>No silica fume</td>
<td>949.3</td>
</tr>
<tr>
<td>UHPC 2</td>
<td>SF -20%</td>
<td>770.9</td>
</tr>
<tr>
<td>UHPC 3</td>
<td>SF -20% + SMF -2%</td>
<td>754.9</td>
</tr>
<tr>
<td>UHPC 4</td>
<td>SF-20%+SMF-2%+SRA-2%</td>
<td>740.7</td>
</tr>
</tbody>
</table>

UHPC 1 & 2 are not fiber reinforced and UHPC 3&4 are 2% fiber reinforced.

Figure 22 shows the compression tests of the different design mixes at 28 days.

UHPC 3&4 has higher compressive strength compared to UHPC 1&2.
Compressive strength increases with increase in time. The minimum compressive strength achieved was 23000 psi in 28 days.

2.2.2.1 Effect of silica fume dosage on compressive strength

Compressive strength of the UHPC at 7 days remained constant irrespective of dosage of silica fume. However with increase in age the compressive strength increased with increase in silica fume dosage. It can be observed from Figure 23 that 28 days compressive strength of UHPC was found to increase with silica fume dosage ranging from 0 to 20%. Further increase in silica fume tend to decrease the compressive strength. 20% silica fume dosage appears optimal from compressive strength standpoint (27).
2.2.2.2 Split tensile strength test

Split tensile test was conducted on 3X6” cylinders for the 4 design mixes used. Design mix 3&4 have the highest split tensile strength as these two design mixes consist of 2% steel fibers. Figure 24 shows the variation in split tensile strength of 4 design mixes.

![Figure 24- Split tensile strength test of different mix designs at 28 days (27)](image)

2.2.2.3 Flexural Strength test

Flexural tensile strength test was performed on 3”X3”X12” beam specimens. Figure 25 shows the results of all the four design mixes. It was observed that by using shrinkage reducer, flexural strength has decreased by 21%.

![Figure 25- Flexural strength of the four different mixes (27)](image)
2.2.3 Research carried out by Islamic university of Gaza

An extensive research was performed at Islamic university of Gaza, Palestine (18) which includes use of naturally available material and different dosages of silica fume and use of steel and polypropylene fibers. Portland cement 42.5 R was used throughout the research and light grey silica fume was used in different dosages. Steel fibers were used 2% by volume and polypropylene fibers were used 0.2 % by volume. The reason behind such less dosage of polypropylene was not to reduce the workability of concrete and increase the durability. Table 16 presents the design mix proportions.

Table 16 – Mix proportions (18)

<table>
<thead>
<tr>
<th>Material</th>
<th>Ingredient/cement content</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement CEM 42.5 R</td>
<td>1.00</td>
</tr>
<tr>
<td>Water to cement ratio (w/c)</td>
<td>0.24</td>
</tr>
<tr>
<td>Silica fume to cement (s/c)</td>
<td>0.15</td>
</tr>
<tr>
<td>Quartz sand to cement (q/c)</td>
<td>1.25</td>
</tr>
<tr>
<td>Superplasticizer to cement</td>
<td>0.03</td>
</tr>
<tr>
<td>Steel fibers to cement</td>
<td>0.16 (2% by volume )</td>
</tr>
<tr>
<td>Polypropylene fibers to cement</td>
<td>0.001</td>
</tr>
</tbody>
</table>

2.2.3.1 Compressive strength

For measuring the compressive strength 100X100X100 mm (4”X4”X4”) size specimens were used and BS 1881 procedure was followed. The rate of loading 0.3 KN/s was maintained. Figure 26, 27&28 shows the effect of silica fume and steel fibers on compressive strength of the concrete at 7, 14 and 28 days respectively.
Figure 26 - Effect of silica fume and steel fibers on compressive strength at 7 days (18)

Figure 27 - Effect of silica fume and steel fibers on compressive strength at 14 days (18)

Figure 28 – Effect of silica fume and steel fibers on compressive strength at 28 days (18)

Figure 29 shows the compressive strength gain over the time. From the figure it can be seen that compressive strength increases with age.
2.2.3.2 Split tensile strength

Split tensile test is the test to measure tensile capacity of the concrete indirectly. It measures the tensile strength of the concrete by compressing a cylinder through a line load applied along its length. The loading conditions produces a high compressive stress immediately below the loading point. But the larger portion of the cylinder, corresponds to its depth, and is subjected to uniform tensile stress acting horizontally. It is estimated that the compressive stress is acting for about 1/6 depth and remaining 5/6 depth is subjected to tension due to poisons ratio. 6”X12” Cylinders were used to measure the split tensile strength of the concrete. Figure 30 shows the effect of silica fume percentage on split tensile strength of the concrete.

Addition of silica fume increases the split tensile strength of the concrete by 13%. It can also be observed that addition of micro steel fibers increase the split tensile capacity of the concrete by 110%.
2.2.3.3 Flexural strength

Effect of silica fume dosage and steel fiber volume fraction study has been done and found that at a 2% steel fiber volume fraction and 15% silica fume dosage higher flexural strength was achieved (18). Figure 31 shows the effect of silica fume and steel fiber dosage on flexural strength of the concrete.

2.2.4 Research carried out by Federal Highway Administration (FHWA)

Benjamin Graybeal and Joseph Hartmann from Federal Highway Administration (28) have studied the strength and durability properties of Ultra high performance concrete.
The research mainly focused on Mechanical properties of the UHPC using different curing regimes.

2.2.4.1 Compressive strength

The standard size 3”X6” cylinders was used for testing the compressive strength. The loading rate was modified to 150 psi/s instead of ASTM C39 35±7 psi/s to speed up the testing process. Preliminary tests assured that change in rate of loading did not affect the strength of the cylinders (28).

Curing effect

Different curing methods were adopted to check the strength variation of the Concrete. Steam curing at 194° F for 48 hours soon after demolding. The ambient air curing means putting the specimens in the laboratory environment from demolding to testing. The tempered stream curing means curing the specimens at 140°F for 48 hours soon after demolding. The final curing is a delayed steam curing, which is same as stream curing except the process is delayed until 15 days after casting. The strength of the stream cured specimens exhibited strength of 25 ksi in 28 days, tempered steam and delayed steam exhibited strengths approximately 10 % lower. The ambient air cured specimens achieved approximately 65% of the strength of the stream cured specimens. Table 17 shows the compressive strength of the cylinders.

<table>
<thead>
<tr>
<th>Curing Method</th>
<th>Samples</th>
<th>Compressive strength (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steam</td>
<td>96</td>
<td>28.0</td>
</tr>
<tr>
<td>Ambient air</td>
<td>44</td>
<td>18.0</td>
</tr>
<tr>
<td>Tempered steam</td>
<td>18</td>
<td>25.2</td>
</tr>
<tr>
<td>Delayed steam</td>
<td>18</td>
<td>24.9</td>
</tr>
</tbody>
</table>
Cylinders and cubes

3”X6” cylinders were used to check the compressive strength of the concrete (28). However some laboratories are beyond the capabilities to do test on cylinders due to preparation of end surface and loading capacity of the machine. With regard to the test machine, capacity for a 3” diameter cylinder for a strength of 31 ksi machine with 220 kips force capacity is required, whereas for the 4” cylinder 390 kip force capacity is required. 2” and 4” cubes were prepared to avoid the problem of the end surface grinding. Table 18 shows the effect of sample geometry on compressive strength.

Table 18- Compressive Strength Comparison of Various Specimen Geometrics (28)

<table>
<thead>
<tr>
<th>Sample geometry</th>
<th>Height</th>
<th>Samples</th>
<th>Compressive strength (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3 in Dia Cylinder</td>
<td>5.8 in</td>
<td>6</td>
<td>29.4</td>
</tr>
<tr>
<td>3 in Dia Cylinder</td>
<td>6.3 in</td>
<td>3</td>
<td>27.2</td>
</tr>
<tr>
<td>4 in Dia Cylinder</td>
<td>7.9 in</td>
<td>5</td>
<td>29.3</td>
</tr>
<tr>
<td>2 in Dia Cylinder</td>
<td>3.8 in</td>
<td>6</td>
<td>29.5</td>
</tr>
<tr>
<td>100 mm Cube</td>
<td>100 mm</td>
<td>5</td>
<td>29.1</td>
</tr>
<tr>
<td>2 in Cube</td>
<td>2.0 in</td>
<td>6</td>
<td>31.1</td>
</tr>
</tbody>
</table>

2.2.4.2 Tensile strength

Concrete tensile capacity is often disregarded due to brittle nature of concrete in tension. UHPC exhibits significantly improved tensile strength both before and after cracking when compared to normal concrete. The tensile capacity permits higher
precracking tensile loads in structural applications. Many test methods have been
developed to measure the tensile strength of the concrete directly or indirectly (28)

**Mortar Briquette Test**

Mortar briquette test is a measure of direct tension test on concrete. The test is
carried out according to AASHTO T132. The test normally involves casting a small
briquette of 1 in² area, neck in the middle and has enlarged ends (28). The enlarged ends
allows the gripping of the specimen. The stream and ambient air cured specimens exhibited
strengths approximately 15% below the overall test program. The delayed stream curing
specimens were 8% below the overall test program on an average. The test results indicated
that tensile strength of the UHPC varies depending on the curing applied. The stream,
tempered stream, delayed stream curing tend to have tensile cracking strengths 1.2 ksi, 1.45
ksi, 1.0 ksi respectively. The ambient cured specimens have the lowest strength of 0.9 ksi
in 28 days. However the strength continue to increase with age 1.1 ksi in 84 days. Figure
32 shows the effect of different curing methods on tensile cracking strength.

![Tensile Cracking Strength](image)

**Figure 32 – Tensile cracking strength (28)**
Relative to conventional concrete, UHPC having steel fiber reinforced are unique, so that they continue to carry the tensile loads even after cracking. This behavior is due to presence of randomly distributed discontinuous fibers. From a structural standpoint, this post cracking tensile strength permits a reduction in other post crack tensile force carrying mechanism (i.e., rebar) provides an increase in ductility and energy dissipation capacity.

The load displacement relationship was recorded during testing of the steam cured Mortar briquette. It was observed in the test that, stiffness of the specimen changes at initial cracking, but it continues to carry the tensile forces through large subsequent displacement as shown in the Figure 33.

![Graph showing load displacement response for a steam briquette specimen](image)

**Figure 33- Load displacement response for a stream briquette specimen (28)**

**Indirect tensile test (split tension):**

ASTM C496 test procedure is used to measure the split tensile capacity of the concrete. The original rate of loading is 100 to 200 psi/min, it is modified to 500 psi/min. Preliminary testing indicated that the change in rate of loading did not influence the test results. Displacement measurement system was attached to the specimen. Figure 34 shows the lateral deflection response of a steam cured specimen.
2.2.5 Research carried out by Michigan Tech University

Premixed ductal composition bags were used for making the UHPC. Doyon mixer with a capacity of 0.65 ft\(^3\) was used for mixing the concrete (7). Table 19 shows the mix proportions used for the experimental work.

Table 19 - Mix proportions (7)

<table>
<thead>
<tr>
<th>Constituents</th>
<th>1.0 yd(^3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Premix</td>
<td>3700 lb</td>
</tr>
<tr>
<td>Water</td>
<td>219 lb</td>
</tr>
<tr>
<td>Superplasticizer</td>
<td>51 lb</td>
</tr>
<tr>
<td>Steel fiber</td>
<td>26.3 lb</td>
</tr>
<tr>
<td>w/c ratio</td>
<td>0.2</td>
</tr>
</tbody>
</table>

Curing Regimes

The specimens were either Air cured (Air), or Thermally Treated (TT). Air cured specimens for testing 3 days strength were demolded 2 to 3 hours before testing. The TT cured specimens were subjected to a 48 hour, 100% humidity, stream treatment at 194°F upon demolding at 3 days. The thermal treatment began with 6 hour of preheating the chamber and the samples were kept in the chamber for 48 hours. At the end of the curing, chamber was made to cool down and samples were taken out. The specimens of delayed
thermal treatment (DTT), and doubly delayed thermal treatment (DDTT) were demolded at 3 days and placed at ambient air temperature until the starting of the thermal treatment at 10, and 24 days. Reid surface grinder was used for the specimen preparation as shown the Figure 35.

Figure 35 - Reid surface grinder (7)

Compressive Strength

3x6” cylinders were used for measuring the compressive strength of the concrete. ASTM C39 procedure for testing the compressive strength of the cylinders with 35±7 psi/s was observed to take 15 minutes for one cylinder. To reduce the time of testing the rate of loading was maintained to be 150 psi/s. Graybeal and Hartmann (2003) conducted research showing that there was no significant difference in strength due to increase in rate of loading (7). Table 20 explains the different curing methods adopted, no of samples tested and mean compressive strength achieved. Figure 36 shows the compressive strength comparison at different ages of curing.
Table 20 - Test results (7)

<table>
<thead>
<tr>
<th>Curing regime</th>
<th>Specimen age at testing (days)</th>
<th>Number of specimens</th>
<th>Mean compressive strength (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Air - cured</td>
<td>3</td>
<td>6</td>
<td>14.4</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>6</td>
<td>19.9</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td>6</td>
<td>22.3</td>
</tr>
<tr>
<td></td>
<td>28</td>
<td>6</td>
<td>23.9</td>
</tr>
<tr>
<td>TT (Thermal Treatment)</td>
<td>7</td>
<td>6</td>
<td>30.3</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td>6</td>
<td>30.1</td>
</tr>
<tr>
<td></td>
<td>28</td>
<td>6</td>
<td>31.1</td>
</tr>
<tr>
<td>DTT (Delayed TT)</td>
<td>14</td>
<td>6</td>
<td>29.7</td>
</tr>
<tr>
<td></td>
<td>28</td>
<td>6</td>
<td>29.9</td>
</tr>
<tr>
<td>DDTT</td>
<td>28</td>
<td>5</td>
<td>29.4</td>
</tr>
</tbody>
</table>

Figure 36- Compressive strength comparison (7)

2.2.5.1 Modulus of Elasticity and Poisons Ratio

Modulus of elasticity is a material dependent property which is described as mathematical relationship between stress and strain. The slope of the elastic portion of the stress strain curve is defined as the modulus of elasticity. The modulus of elasticity is used in the design calculations to predict the deflection behavior of the element so that the design can satisfy the specified limit states. Testing modulus of elasticity is time consuming process and requires testing jigs and cylinders with flat top and bottom surfaces. Efforts
have undertaken to develop the relationship between compressive strength and modulus of elasticity (7).

ACI 318 presents an equation which relates the 28 days compressive strength ($f'_c$) of the normal concrete, for concrete with unit weight ($w_c$) of 90 to 150 pcf.

$$Ec = w_c^{1.5} \times 33 \times \sqrt{f'_c} \text{ (psi)} \quad \ldots \ldots \quad (6)$$

However HPC has higher compressive strength compared to normal strength concrete. ACI Committee 363 produced a relationship for higher strength concrete with compressive strength from 3,000 to 12,000 psi.

$$Ec = 40,000 \times \sqrt{f'_c} + 1.0 \times 10^6 \text{ (psi)} \quad \ldots \ldots \quad (7)$$

The two equations mentioned do not apply for the concrete which has higher compressive strength above 12,000 psi. Additionally, the interim recommendations for UHPC, which was requested by the Association Française de Génie Civil (AFGC) and published by Service d’études Techniques des Routes et Autoroutes (SETRA), does not have an equation relating compressive strength to modulus of elasticity.

A relationship between compressive strength and modulus of elasticity of the UHPC from the work done at the Cattenom nuclear power plant on 196 cylindrical specimens with diameter of 2.76 in is given as

$$Ec = 262,000 \times 3 \sqrt{f'_{ATT}} \quad \ldots \ldots \quad (8)$$

Where $f'_{ATT}$ = compressive stress of UHPC after thermal treatment.

Research conducted at Iowa State University by Sritharan et al (2003) on five 3X6” cylinder specimen produced an equation.

$$Ec = 50,000\sqrt{f'_{ATT}} \text{ (psi)} \quad \ldots \ldots \quad (9)$$

Where $f'_{ATT}$ = compressive stress of UHPC after thermal treatment.
Graybeal (2005) developed a relationship using total of 148 samples undergoing different curing regimes stream, air, tempered stream, delayed stream. The relationship proposed for the compressive strength of the concrete between 4 to 28 ksi.

\[ Ec = 46,200 \times \sqrt{f'_c} \text{ (psi)} \] 

Work done at Michigan Tech by Kollmorgen (2004) resulted an equation relating the compressive strength and modulus of elasticity. Twenty four cylindrical specimens of 2”X4”, 3”X6”, and 4”X8” were used to determine the relationship with an acceptable range of 5 to 30 ksi.

\[ E_c = 351,000 \times \frac{3.14}{\sqrt{f'_{\text{ATT}}}} \text{ (psi)} \] 

Where \( f'_{\text{ATT}} \) = compressive stress of UHPC after thermal treatment

2.2.5.2 First Crack Flexural Strength and Flexural Toughness

Studies conducted by Michigan Tech University (7) on flexural strength of the UHPC. Testing was conducted on 2”X2”X11.25” beam specimens not on standard ASTM C 1018 4X4X14”. Graybeal pointed that, UHPC elements subjected to flexural forces are not likely to be 4” thick. ASTM C1018 states that the cross section of the specimen needs to be 3 times the fiber length. ASTM C1018 standard test method for flexural toughness and first strength of fiber reinforced concrete was used to determine the flexural strength and first crack strength of Ductal. Test consist of loading the small prism at the three points, and recorded the load and deflection so that the data can be analyzed to give flexural cracking stress, toughness and flexural strength of the fiber reinforced concrete. Table 21 explains the first crack load of different curing adopted specimens. Figure 37 shows the flexural strength test set up according to ASTM C1018.
Table 21 – First crack load and crack deflection details (7)

<table>
<thead>
<tr>
<th>Curing regime</th>
<th>Specimen age (days)</th>
<th>No of samples</th>
<th>Sample mean first crack deflection (in)</th>
<th>Sample mean first crack load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Air</td>
<td>28</td>
<td>12</td>
<td>0.00179</td>
<td>1.23</td>
</tr>
<tr>
<td>28 – No Fiber</td>
<td>3</td>
<td>0.00205</td>
<td>1.36</td>
<td></td>
</tr>
<tr>
<td>TT</td>
<td>28</td>
<td>12</td>
<td>0.00240</td>
<td>1.7</td>
</tr>
<tr>
<td>28 – No Fiber</td>
<td>3</td>
<td>0.00275</td>
<td>1.82</td>
<td></td>
</tr>
<tr>
<td>DTT</td>
<td>28</td>
<td>12</td>
<td>0.00273</td>
<td>1.93</td>
</tr>
<tr>
<td>28 – No Fiber</td>
<td>3</td>
<td>0.00307</td>
<td>1.98</td>
<td></td>
</tr>
</tbody>
</table>

Figure 37 – Flexural strength test (7)

Figure 38 shows the load deflection curve for the flexural specimens. From the figure it can be seen that after the 1st crack, beam was able to withstand more load.
2.2.6 Research Conducted at University Sains Malaysia

The interfacial bonding between deteriorated concrete structures and newly overlay repair material is the one of the most important factor for the structural functionality and durability. The research was conducted to examine experimentally the mechanical properties and permeability characteristics of the interface between normal concrete (NC) and Ultra-High Performance Fiber Reinforced Concrete (UHPFRC) as a repair material. The mechanical interfacial bond characteristics were assessed using slant shear and splitting tensile strength to quantify the difference of different roughened surfaces. Permeability characteristics were evaluated by rapid chloride permeability, gas and water permeability tests. In the field of retrofitting and strengthening of the concrete structures, the need to place new concrete material next to the old concrete would often arise, in these cases bond strength between new concrete and old concrete generally presents a weak link in the repaired structures (29). Table 22 shows the design mix proportions of normal strength concrete and ultra-high performance fiber reinforced concrete.
Table 22 - Different grades of concrete for evaluating the bond strength (29)

<table>
<thead>
<tr>
<th>Concrete type (kg/m³)</th>
<th>Normal concrete (NC)</th>
<th>UHPFRC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>400</td>
<td>768</td>
</tr>
<tr>
<td>Coarse aggregate (max 12.5 mm)</td>
<td>930</td>
<td>-</td>
</tr>
<tr>
<td>River sand</td>
<td>873</td>
<td>-</td>
</tr>
<tr>
<td>Mining sand</td>
<td>-</td>
<td>1140</td>
</tr>
<tr>
<td>Silica fume</td>
<td>-</td>
<td>192</td>
</tr>
<tr>
<td>Steel fiber</td>
<td>-</td>
<td>157</td>
</tr>
<tr>
<td>Superplasticizer</td>
<td>4</td>
<td>40</td>
</tr>
<tr>
<td>Water</td>
<td>200</td>
<td>144</td>
</tr>
<tr>
<td>Total</td>
<td>2407</td>
<td>2441</td>
</tr>
<tr>
<td>W/B</td>
<td>0.5</td>
<td>0.15</td>
</tr>
<tr>
<td>Cube strength</td>
<td>45 MPA</td>
<td>170 MPA</td>
</tr>
<tr>
<td>Split cylinder tension test</td>
<td>2.75 MPA</td>
<td>15.3 MPA</td>
</tr>
</tbody>
</table>

2.2.6.1 Slant Shear test

Specimens for slant shear tests (29) were prepared using normal concrete and at the 3rd day, they were dried and went through surface preparation/roughening process. Five different surface textures were used, (i) without any surface preparation i.e. as casted (AC) (ii) sand blasted (SB) purposely exposing the aggregates (iii) wire brushed (WB) without exposing the aggregates (iv) drilled holes (DH) each hole having 10 mm diameter and 5 mm depth (v) grooved (GR) with 10 mm width and 5 mm depth. All the surface prepared specimens are shown in the Figure39. After the surface preparation, the samples were taken into the water tank to cure them for 28 days and then taken out from the water tank and left to dry for 2 months, so the total duration before casting with UHPFRC is 3 months. Before casting the UHPFRC onto the NC substrates the surface of the roughened surface samples were moistened for 10 minutes and then they were dried.
The composite specimens were cured by steam for 48 hours at a temperature of 90°C and 100% relative humidity, subsequently they were cured in water, maintained at a temperature of 27°C until the day of testing. Figure 40 shows the slant shear test preparation. Figure 41 shows the split tensile test set up.

Figure 39- Different surface preparation of the samples (29)

Figure 40 – Slant shear test (29)
2.2.6.1 Results & discussion

The experimental results showed the interfacial bonding for all the surface roughened specimens were good and strong enough as the interfacial failure mostly occurred after the substrate experienced some degree of damage. The interfacial bond strength observed to increase with age which could be linked to the hydration of cement and pozzolanic reaction between cement and silica fume producing calcium silicate hydrates and increasing the strength of UHPFC as well as the interfacial bond strength of the composite. In the case of no surface preparation, the measured shear strength were 8.18, 8.47, 8.68 MPA in 3, 7, 28 days respectively. In the case of sand blasted (SB) surface specimen, it was recorded that the highest 28 days shear strength was 17.8 MPA. Split cylinder tensile strength of the composites increased slightly with age. No surface prepared samples reported 1.67, 1.82, 1.85 MPA in 3, 7, 28 days. Sand blasted (SB) composite specimens achieved the highest split tensile strength of 3.79 MPA in 28 days. (29). Figure 42 shows the slant shear strength test results. Figure 43 shows the split tensile bond strength.
Figure 42- Bond strength of the different surface prepared specimens (29)

Figure 43- Split tensile bond strength (29)

Table 23 describes the ACI concrete repair limits of interfacial bond strength. Table 24 explains the bond strength requirements.
Table 23 - Minimum acceptable bond strength (ACI Concrete Repair Guide) (29)

<table>
<thead>
<tr>
<th>Days</th>
<th>Interfacial Bond strength (S) (MPA)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2.76-6.9</td>
</tr>
<tr>
<td>7</td>
<td>6.9-12.41</td>
</tr>
<tr>
<td>28</td>
<td>12.41-20.68</td>
</tr>
</tbody>
</table>

Table 24 - Quantitative bond quality in terms of bond strength (29)

<table>
<thead>
<tr>
<th>Bond quality</th>
<th>Bond strength T (MPA)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Excellent</td>
<td>≥2.1</td>
</tr>
<tr>
<td>Very good</td>
<td>1.7-2.1</td>
</tr>
<tr>
<td>Good</td>
<td>1.4-1.7</td>
</tr>
<tr>
<td>Fair</td>
<td>0.7-1.4</td>
</tr>
<tr>
<td>Poor</td>
<td>0-0.7</td>
</tr>
</tbody>
</table>

The bond strength in the slant shear test was very good, as the interfacial damage occurred after the damage of NC substrate. The NC substrate firstly exhibited cracking and crushing prior to the interfacial failure.

The results of split tensile strength specimen’s shows that the failure mostly occurred due to NC substrate, it means that the UHPFC bonded very strongly with the NC substrate and behaved almost monolithically. Figure 44 shows the different failure patterns.
Type A failure = Interface failure
Type B failure = Interface failure and substrate cracks
Type C failure = Interface failure and substrate fracture
Type D failure = Substratum failure

2.2.7 Research carried out by University of Hong Kong

Extensive research was conducted at the University of Hong Kong (30) to study the properties of Reactive Powder Concrete (RPC) and High Performance Concrete (HPC) and
their performance at different temperatures. Different properties have been examined such as compressive strength, split tensile strength and modulus of elasticity of both the concretes. Two different concrete mix designs were used with the same water binder ratio. Table 25 gives the details of the mix proportions of reactive powder concrete and high performance concrete.

Table 25- Details of RPC and HPC mix designs (30)

<table>
<thead>
<tr>
<th>Type of concrete mix</th>
<th>RPC</th>
<th>HPC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Water/Binder ratio</td>
<td>0.24</td>
<td>0.24</td>
</tr>
<tr>
<td>Cement : kg/m³</td>
<td>761</td>
<td>761</td>
</tr>
<tr>
<td>Silica fume : kg/m³</td>
<td>247</td>
<td>247</td>
</tr>
<tr>
<td>20 mm aggregates : kg/m³</td>
<td>Nil</td>
<td>488</td>
</tr>
<tr>
<td>10 mm aggregates : kg/m³</td>
<td>Nil</td>
<td>244</td>
</tr>
<tr>
<td>River sand : kg/m³</td>
<td>Nil</td>
<td>314</td>
</tr>
<tr>
<td>Quartz sand : kg/m³</td>
<td>1090</td>
<td>Nil</td>
</tr>
<tr>
<td>Crushed quartz : kg/m³</td>
<td>226</td>
<td>Nil</td>
</tr>
<tr>
<td>Water : kg/m³</td>
<td>244</td>
<td>244</td>
</tr>
<tr>
<td>Superplasticizer : kg/m³</td>
<td>15</td>
<td>15</td>
</tr>
</tbody>
</table>

Conventional mixing procedure is adopted for the HPC concrete and a different mixing procedure is adopted for RPC as it contains very finer particles. Main focus of the research was to compare mechanical properties of both the concretes.

RPC Mixing procedure, constitutes dry mixing the powders (cement, silica fume, quartz powder, silica sand) for about 3 minutes and then addition of half the volume of water containing half amount of superplasticizer and mixed for 3 mainland finally the remaining water and superplasticizer was added and mixed for 10 minutes. The extended time of mixing required to disperse silica fume fully (35). The specimens were compacted and demolded after 24 hours and some samples were placed in the oven at a temperature of 250°C for 16 hours. 4” cubes were used for compressive strength and 4”X8” Cylinders were used for measuring the split tensile strength of the concrete. Tests were done at
different temperatures and in different curing periods. It was observed that the strength of RPC is higher than HPC over the time at the same water binder ratio. The reason behind the higher strength of RPC is due to enhancement of homogeneity by elimination of coarse aggregates and leads to the high packing density of solid particles of the RPC. Table 26 describes the results of the tests performed.

Table 26 - Results and Discussion (30)

<table>
<thead>
<tr>
<th>Concrete Mix</th>
<th>Temperature °C</th>
<th>Duration: Days</th>
<th>Compressive strength (MPA)</th>
<th>Split tensile strength (MPA)</th>
<th>Modulus of Elasticity :GPA</th>
</tr>
</thead>
<tbody>
<tr>
<td>RPC</td>
<td>27</td>
<td>7</td>
<td>95.2</td>
<td>7.02</td>
<td>41</td>
</tr>
<tr>
<td></td>
<td>27</td>
<td>14</td>
<td>117</td>
<td>7.32</td>
<td>43</td>
</tr>
<tr>
<td></td>
<td>27</td>
<td>28</td>
<td>134</td>
<td>7.71</td>
<td>45</td>
</tr>
<tr>
<td></td>
<td>27</td>
<td>56</td>
<td>138</td>
<td>8.07</td>
<td>45</td>
</tr>
<tr>
<td>HPC</td>
<td>27</td>
<td>7</td>
<td>83.4</td>
<td>5.25</td>
<td>28.5</td>
</tr>
<tr>
<td></td>
<td>27</td>
<td>14</td>
<td>90.3</td>
<td>5.45</td>
<td>34</td>
</tr>
<tr>
<td></td>
<td>27</td>
<td>28</td>
<td>91.8</td>
<td>5.45</td>
<td>38</td>
</tr>
<tr>
<td></td>
<td>27</td>
<td>56</td>
<td>100</td>
<td>5.53</td>
<td>38</td>
</tr>
<tr>
<td>RPC</td>
<td>100</td>
<td>16 h</td>
<td>116</td>
<td>7.24</td>
<td>39.1</td>
</tr>
<tr>
<td></td>
<td>100</td>
<td>48 h</td>
<td>127</td>
<td>7.31</td>
<td>41.5</td>
</tr>
<tr>
<td></td>
<td>250</td>
<td>16 h</td>
<td>195</td>
<td>7.47</td>
<td>42.0</td>
</tr>
<tr>
<td></td>
<td>250</td>
<td>48 h</td>
<td>200</td>
<td>7.53</td>
<td>43.7</td>
</tr>
</tbody>
</table>

Figure 45 shows the compressive strength RPC and HPC at different curing periods. Due to presence of larger size aggregates High Performance Concrete could not able achieve higher strength at the same water to cement ratio as RPC. Due to removal of aggregate there was an increase of 45% strength at 28 days under similar curing conditions.
Figure 45- Compressive strength of RPC & HPC samples at different curing periods (30)

Figure 46 shows the split tensile strength of RPC and HPC respectively. From figure it can be seen that the RPC has 41% higher splitting tensile strength compared to HPC. At the higher loads aggregates becomes the weakest link in the concrete.

Figure 46- Split tensile strength of RPC and HPC samples at different curing periods (30)

Figure 47 shows the static modulus of elasticity of RPC and HPC samples at different curing ages. It can be seen that modulus of elasticity is always higher than that of HPC. Static modulus of elasticity decreases when the interfacial behavior between aggregates and paste gets weekend.
Figure 47- Modulus of elasticity of RPC & HPC samples at different curing periods (30)

Figure 48 shows the stress strain graph of RPC and HPC. From the figure we can see that RPC is steeper and remains linear up to the higher ultimate strength. Both the curves end abruptly, descending parts of the curves show a very steep slope, indicating RPC and HPC are brittle in nature. The solution to overcome brittleness is incorporation of fibers.

Figure 48- Stress strain curves of RPC and HPC (30)

2.2.8 Research carried out by Islamic University of Gaza & University Sains Malaysia

An extensive study has been conducted at Islamic University of Gaza (31) on the effect of different curing conditions on mechanical properties of UHPC concrete. Six different curing conditions were used in the research work. It is defined that “curing is the name given to procedures used to enhance the hydration of the cement and consist of control of temperature and of the moisture and of moisture movement from and into
concrete”. Curing leads to better strength development and allows more water to be available for the hydration reaction of the concrete’s cement paste. Better curing conditions increase the compressive strength of the concrete and improves the durability properties as well. Table 27 shows the design mix proportions of UHPC used for experimental work.

Table 27- The mix design used is shown below (31)

<table>
<thead>
<tr>
<th>Material</th>
<th>Amount (Kg/m³)</th>
<th>Percent by weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>Portland cement</td>
<td>825</td>
<td>33.60</td>
</tr>
<tr>
<td>Fine sand</td>
<td>1140</td>
<td>46.43</td>
</tr>
<tr>
<td>Silica fume</td>
<td>181</td>
<td>7.37</td>
</tr>
<tr>
<td>Super plasticizer</td>
<td>45</td>
<td>1.83</td>
</tr>
<tr>
<td>Steel fibers</td>
<td>90</td>
<td>3.66</td>
</tr>
<tr>
<td>Water</td>
<td>174</td>
<td>7.08</td>
</tr>
<tr>
<td>Water binder ratio</td>
<td>174/1006</td>
<td>0.17</td>
</tr>
</tbody>
</table>

The procedure of mixing was, sand was washed and sieved, silica fume and cement placed in the mixer in the sequence manner. Dry materials were well mixed and later water and superplasticizer were added.

2.2.8.1 Curing regimes

Six different curing regimes were used for studying the mechanical properties:

- Spraying specimens with water (fog room)
- Immersing in water
- Normal curing
- Boiling water curing method
- Stream cured after one day casting
- Stream cured after two days casting
In the fog curing method, specimens were tested after 3, 7, 14, 28 days. 

In immersed curing method specimens were demolded after 24 hours till it gets hardened and put in the water till the time of testing.

In normal curing method after 2 hours demolding specimens were placed in ambient weather with temperature near to 28°C. Specimens were tested for 3, 7, 14, 28 days.

In boiling water curing method, the specimens were moist cured in laboratory for 23 hours. After elapsing the 23 hours the specimens were transferred into curing for 3.5 hours maintained at 100°C temperature.

In stream cured process, specimens left for 24 hours without curing then later applied the curing treatment includes streaming UHPC at 90°C for 60 hours and then specimens were put in the water tank.

In the last curing method, specimens the specimens were left for 2 days instead of 1 day and then cured at 90°C for 60 hours and then specimens were put in the water tank.

2.2.8.2 Compressive strength

The specimens of concrete were 4”X4”X4” (100X100X100 mm) and rate of loading applied was 0.2 -0.4KN/s. Total of 3 specimens were tested at 3, 7, 14, 28 days. British standard code BS 1881-116:1983 was followed to measure the compressive strength of the concrete. Table 28 gives the compressive strength results of different curing procedure adopted specimens at different days. Figure 49 shows the graphical representation of compressive strength results.
Table 28 - Compressive strength given in Mpa (31)

<table>
<thead>
<tr>
<th>Curing condition</th>
<th>3 days</th>
<th>7 days</th>
<th>14 days</th>
<th>28 days</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stream curing one day delay</td>
<td>164.45</td>
<td>169.59</td>
<td>171.86</td>
<td>173.6</td>
</tr>
<tr>
<td>Stream curing two days delay</td>
<td>-</td>
<td>166.35</td>
<td>168.58</td>
<td>170.54</td>
</tr>
<tr>
<td>Boiling curing</td>
<td>163.63</td>
<td>167.74</td>
<td>170.39</td>
<td>172.48</td>
</tr>
<tr>
<td>Fog room curing</td>
<td>119.31</td>
<td>137.62</td>
<td>150.49</td>
<td>158.28</td>
</tr>
<tr>
<td>Water curing</td>
<td>119.86</td>
<td>139.74</td>
<td>152.36</td>
<td>159.86</td>
</tr>
<tr>
<td>Normal curing</td>
<td>117.17</td>
<td>132.93</td>
<td>144.67</td>
<td>152.63</td>
</tr>
</tbody>
</table>

Compressive strength proportionally increase with age. In stream curing UHPC has relatively higher compressive strength in 28 days. In stream curing two delay process and in boiling water cured methods compressive strengths of 170.54 and 172.48 Mpa were recorded respectively. It can be said that type of curing has lot of effect on compressive strength. Stream and boiling curing methods are used to accelerate the rate of strength development of concrete. Normal curing method, which is an ambient air cured method has the lowest compressive strength among all the curing practices.

![Figure 49 – Compressive strength vs curing regimes (31)](image)

2.2.8.3 Flexural strength

Flexural strength represents one of the most important property of concrete which tells us how the concrete behaves in real world. For measuring the flexural strength
70X70X280 mm (2.75”X2.75”X11”) size specimens were used. The specimens were tested for 3, 7, 14, 28 days. Table 29 shows the results of different curing processed specimens.

Table 29- Results of different curing regimes (31)

<table>
<thead>
<tr>
<th>Curing condition</th>
<th>3 days</th>
<th>7 days</th>
<th>14 days</th>
<th>28 days</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stream curing one day delay</td>
<td>25.72</td>
<td>26.43</td>
<td>27.32</td>
<td>27.92</td>
</tr>
<tr>
<td>Stream curing two days delay</td>
<td>-</td>
<td>25.64</td>
<td>26.32</td>
<td>26.86</td>
</tr>
<tr>
<td>Boiling curing</td>
<td>25.27</td>
<td>25.83</td>
<td>26.76</td>
<td>27.42</td>
</tr>
<tr>
<td>Fog room curing</td>
<td>16.68</td>
<td>18.75</td>
<td>20.34</td>
<td>21.64</td>
</tr>
<tr>
<td>Water curing</td>
<td>16.95</td>
<td>19.31</td>
<td>21.16</td>
<td>22.21</td>
</tr>
<tr>
<td>Normal curing</td>
<td>15.93</td>
<td>17.62</td>
<td>18.71</td>
<td>19.69</td>
</tr>
</tbody>
</table>

The flexural strength of stream cured beams have the flexural strength of 27.92 Mpa in 28 days which is the highest among all the curing conditions and for air cured 19.69 Mpa which is the lowest. Figure 50 shows the graphical representation of flexural strength results.

![Figure 50 – Curing method vs flexural strength (31)](image)

From the two experiments done, we can see that curing type and curing period has lot of effect on mechanical properties of UHPC. Increase of temperature method effects the microstructure of the concrete.
2.2.9 Flexural behavior of fiber reinforced concrete beams

The approach to find the moment carrying capacity of the reinforced concrete beams with the fibers is different to conventional reinforced concrete beam. ACI 544.4R-88, procedure is followed to find the moment carrying capacity where the parabolic compressive stress zone is divided into two parts, rectangular and triangular (51). Beam is designed as under reinforced concrete beam in which steel yields first, at the same time the beam is reinforced with steel fibers as well. Figure 51 shows the stress strain profile concrete with fiber content.

\[
M_u = T_s \times Z_1 + T_f \times Z_2 \tag{12}
\]

Where \(T_s\) = tensile force carried by steel bars

\(T_f\) = tensile force carried by steel fibers

\(Z_1\) \& \(Z_2\) = respective lever arm distances

\[
T_s = A_s \times f_y \tag{13}
\]

\[
T_f = \sigma_t \times b \times (h-c) \tag{14}
\]
\( \sigma_t = \text{ultimate tensile strength of fiber reinforced concrete and is greatly influenced by the properties and contents of fibers.} \)

Determination of \( Z_1 \) and \( Z_2 \)

\( C_1 = \text{Compressive force corresponding to the area of the rectangular compressive stress block} \)

\( C_2 = \text{compressive force corresponding to the area of triangular compressive stress block} \)

\[
C_1 = 0.85 f'c x 0.80 c^*b = 0.68 f'c c^*b \hspace{1cm} (15)
\]

\[
y_1 = (0.80 * c)/2 = 0.4^*c \hspace{1cm} (16)
\]

\[
C_2 = (1/2)^*(0.85x f'c x 0.20c) b = 0.085 f'c cb \hspace{1cm} (17)
\]

\[
y_2 = (1/3) (0.20xc) +0.80c = 0.867 c \hspace{1cm} (18)
\]

\[
\Sigma C = C_1 + C_2 = 0.765 f'c cb \hspace{1cm} (19)
\]

\[
y'c = \frac{C_1 y_1+C2y2}{\Sigma C} \hspace{1cm} (20)
\]

\[
Z_1 = d- y'c \hspace{1cm} (21)
\]

\[
Z_2 = (\frac{h-c}{2}) + (c- y'c) \hspace{1cm} (22)
\]

Depth of the neutral axis “c” is determined by the following equation containing \( \sigma_t \).

Equating the compression equal to tension we get the depth of the neutral axis.

\[
C = T_s + T_f \hspace{1cm} (23)
\]

Depth of the neutral axis

\[
c = \frac{\sigma t bh + As fy}{0.765 f'cb + \sigma t b} \hspace{1cm} (24)
\]

The ultimate tensile strength of the fiber reinforced concrete is calculate by followed equation

\[
\sigma_t = \alpha_0 x V_f x \sigma_f x x_b \hspace{1cm} (25)
\]
Where $\alpha_o = \text{orientation factor and is equal to 0.41 (53)}$

$\alpha_b = \text{bond efficiency factor whose value varies from 1 to 1.2 depending on fiber characteristics. For straight fibers value is taken as 1}$

$V_f = \text{volume fraction of the fibers}$

$\sigma_f = \text{tensile strength of the fibers (1000MPa (145 ksi))}$

2.2.10 Impact resistance test

Mechanical properties of the concrete are not sufficient to judge the performance of the concrete, apart from static loads structures are subjected dynamic loads such as blast, earthquake etc.

Impact load on reinforced concrete structures

The large scale tests and computer simulations are usually employed to study the structure under impact loads. The performance of concrete structures are different under impact compared to static loading. Figure 52 shows the idealized impact load profile.

![Figure 2.2 Idealized impact loading profile](image-url)

Figure 52- Idealized impact load profile (55)
There is no well-defined unified approach for determining the failure mode or the resistance of reinforced concrete structures under dynamic loading. There are soft impacts and hard impacts, in hard impact the impacting object shows little damage whereas in soft impacts impacting objects shows significant damages. The effect of impacts on reinforced concrete are summarized as (i) the localized failure mechanisms and most energy absorption is mainly local deformations (ii) impact energy absorbed due to deformations in bending and/or shear energy in global structures (iii) combined local and global failure mechanism (55).

Field impact tests are used to define the failure mode of the reinforced concrete structures. Several impacts are reported by the researchers such as pendulum impact tests, vertical drop impact test.

Figure 53 shows the different types of failures when the specimens subjected to impact forces.

![Figure 53 - Missile impact effects on concrete targets (55)](image)

(a) Penetration, (b) cone cracking, (c) spalling (d) cracks on (i) proximal face (ii) distal face, (f) perforation and (g) overall target structure response

Estimation of the ultimate capacity of the slabs using yield line theory:
Yield line theory (54)

- Prior to the cracking, the distribution of bending moments is according to linear elastic theory
- After cracking, the distribution of bending moments changes due to decrease in flexural rigidity of the cracked portion
- With further loading, yielding of tension steel occurs, and the slab undergoes a redistribution of bending moments
- As the load on the slab is further increased, the lines of intense cracking across which steel has yielded will propagate, until the sufficient lines are formed to collapse the slab. These lines are referred to as yield lines
- First yielding of the tension steel generally occurs at the point of maximum bending moment, but the final cracking pattern depends on many factors such as reinforcement arrangement, type of loading
- Once a mechanism has formed, the slab segments between the yield lines are plane; thus all additional deformations take place at the yield lines

In the presence of concentrated loads, concentrations of yield lines around the loaded are likely to occur. Generally, for concentrated loads, yield line patterns involving radial lines are likely to be more critical than patterns involving large triangular slab segments. Circular fan failure is likely to occur when major concentrated loads are present. Figure 54 shows the different types of impact failures.
For a yield line that runs at right angles to the reinforcement, the ultimate resistance is calculated by. Figure 55 stress profile of the rectangular section to calculate the moment carrying capacity. Figure 56 shows the segment of a fan failure showing moment and rotation.

\[ \text{Mu} = A_s f_y \times (d - 0.6A_s x f_y/f_c) \] ................. (26)

Where \( A_s \) = total area of tension reinforcement

\( f_y \) = yield strength of steel

\( d \) = effective depth

\( f_c \) = compressive strength of the concrete

Work done by external concentrated load = \( P \delta \)

The expression for the ultimate load gives = \( 2\pi \text{Mu} \)
Energy absorption capacity for circular fan failure

Internal work done on slab sector is given by \( L_i = 2\pi \times M_u \times \delta \) .................................. (27)

Considering the external work done by external concentrated load is = \( P \times \delta \)

The expression for ultimate load is given by \( P_u = 2\pi \times M_u \) ....................................... (28)

The ideal ultimate resistance per unit width is calculated by

\[
M_u = A_s f_y x (d-0.6A_s x f_y/f_c)
\]

Impact energy from analysis using yield line = \( P_u \times \delta \)
Impact energy from test = m.g.h ................................. (29)

Where \( m = 300 \text{ lb} \) \((\frac{300}{2.20} = 136.6 \text{ kg})\)

\( G = 9.81 \text{ m.s}^{-2}, \ H = \text{height in meter} \)

2.2.9 Durability tests on UHPC

Apart from high compressive strength, high flexural tensile strength, UHPC possess good durability as well.

Chloride Ion Penetration

Mehta and Monteiro (2005) define permeability as the ease with which a fluid under pressure flows through a solid (32). A concrete with higher permeability is more susceptible to chloride ingress and eventually leads to corrosion of reinforcing steel. Chloride ion migration through concrete by means of capillary absorption, diffusion. However previous research demonstrated that UHPC exhibited almost no permeability and was not susceptible to chloride ingress. The lower water binder ratio and densely packed matrix of UHPC contribute to the impermeability of UHPC. Permeability testing demonstrated that UHPC has an oxygen permeability of less than \(1.6 \times 10^{-15} \text{ in}^2\) (AFGC 2002) while O’Neil et al (1997) reported water absorption of \(7.1 \times 10^{-5} \text{ in}^2\) on the other hand High Performance Concrete (HPC) has water absorption of \(49.7 \times 10^{-5} \text{ in}^2\) Cherezy et al (1995) (7) used mercury intrusion to demonstrate that the porosity of RPC is less than 9% in volume of the pore diameter ranges from \(1.48 \times 10^{-7} \text{ in}\) to \(3.74 \times 10^{-3} \text{ in}\). Another method to determine whether a concrete is susceptible to chloride ingress an applied electrical across a specimen load cell to determine concrete’s conductance (ASTM C 1202). Research conducted by Graybeal (2006) demonstrated that UHPC had negligible chloride ion penetration when thermally treated and only low penetration when not thermally treated.
Graybeal also demonstrated that steel fibers did not contribute to the short circuit effect during UHPC testing. Toutanji et al (1998) revealed that adding 0.75” polypropylene fibers increased the permeability of UHPC whereas adding shorter fiber 0.5” length reduced the permeability of UHPC (33). Furthermore they also stated that addition of silica fume greatly reduced the chloride ion penetration.

2.2.9.1 Experimental investigation

7 day thermally treated, 28 days air treated and 28 days thermally specimens were used for measuring the chloride ion penetration. Thermally treated (TT), specimens were subjected to 48 hour, 100% humidity stream treatment at 194°F upon demolding at 3 days and specimens were taken out of the heating chamber and placed at ambient air till the time of testing. Table 30 presents the results of rapid chloride penetration.

Table 30 - Michigan Tech Rapid Chloride Penetration Summary Data (7)

<table>
<thead>
<tr>
<th>Curing regime</th>
<th>Age at Testing (days)</th>
<th>No. specimens</th>
<th>Charge passed (coulombs)</th>
<th>Chloride Ion Penetrability</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Average</td>
<td>Standard deviation</td>
</tr>
<tr>
<td>Air</td>
<td>28</td>
<td>4</td>
<td>75</td>
<td>15</td>
</tr>
<tr>
<td>TT</td>
<td>7</td>
<td>3</td>
<td>10</td>
<td>1.5</td>
</tr>
<tr>
<td>TT</td>
<td>28</td>
<td>4</td>
<td>15</td>
<td>3.5</td>
</tr>
</tbody>
</table>

Graybeal (2006) (34) has also done the test on chloride ion penetrability on the specimens subjected to different curing regimes and his conclusion was heat treated samples has negligible ion penetrability when compared to air cured specimens. Table 31 presents the results of rapid chloride penetration conducted by Graybeal.
Table 31 - Results of Graybeal (7)

<table>
<thead>
<tr>
<th>Curing regime</th>
<th>Age at testing (days)</th>
<th>No specimens tested</th>
<th>Charge passed (coulombs)</th>
<th>Chloride Ion Penetrability</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Average</td>
<td>Standard deviation</td>
</tr>
<tr>
<td>Stream</td>
<td>28</td>
<td>3</td>
<td>18</td>
<td>1</td>
</tr>
<tr>
<td>Untreated</td>
<td>28</td>
<td>2</td>
<td>360</td>
<td>2</td>
</tr>
<tr>
<td>Untreated</td>
<td>56</td>
<td>3</td>
<td>76</td>
<td>18</td>
</tr>
<tr>
<td>Tempered stream</td>
<td>28</td>
<td>3</td>
<td>39</td>
<td>1</td>
</tr>
<tr>
<td>Tempered stream</td>
<td>56</td>
<td>3</td>
<td>26</td>
<td>4</td>
</tr>
<tr>
<td>Delayed stream</td>
<td>28</td>
<td>3</td>
<td>18</td>
<td>5</td>
</tr>
</tbody>
</table>

2.2.9.2 Limitations of Permeability by FHWA

RCP limits specified by FHWA for prediction of chloride ion penetration are (27)

>4000: “high permeability”

2000-4000: “moderate permeability”

1000-2000: “low permeability”

100-1000: “very low permeability”

<100: “negligible permeability”
3.1 Introduction

The details of the experimental work such as materials used, mix designs, formwork prepared, and testing of the samples will be presented in this chapter. At the initial stages of the work, different trail mixes were conducted to determine the optimum design mix. The optimum design mix was used to carry out for further experimentation.

3.2 Ingredients of Ultra-High Strength Fiber Reinforced Concrete

After thorough investigation and based on research conducted in the past, it was concluded that use of bigger size gravel were not suitable for Ultra-High Performance Concrete. Two different types of the cements (type I and type III) un-densified silica fume, water, High range water reducing agents, and fibers were used in the concrete as shown in the Figure 57. Powered water reducing agent were used and the properties of which would be discussed in further chapters.
3.2.1 Cement

Ordinary Portland cement (OPC) from CEMEX Company, and type III cement from ESSROC Company satisfying ASTM C150 / C150M-12 (56) was used. The chemical composition of the cement is listed in the Table 32.

Table 32 - Chemical composition of different types of cements

<table>
<thead>
<tr>
<th>Oxides</th>
<th>Type 1 cement (in %)</th>
<th>Type 111 cement (in %)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Silicon dioxide</td>
<td>19.2</td>
<td>19.55</td>
</tr>
<tr>
<td>Aluminum oxide</td>
<td>4.7</td>
<td>4.98</td>
</tr>
<tr>
<td>Ferric oxide</td>
<td>3.0</td>
<td>3.08</td>
</tr>
<tr>
<td>Calcium oxide</td>
<td>62.5</td>
<td>64.85</td>
</tr>
<tr>
<td>Magnesium oxide</td>
<td>4.4</td>
<td>0.59</td>
</tr>
<tr>
<td>Sodium oxide</td>
<td>0.84</td>
<td>-</td>
</tr>
<tr>
<td>Potassium oxide</td>
<td>-</td>
<td>0.79</td>
</tr>
<tr>
<td>Sulfur trioxide</td>
<td>3.2</td>
<td>2.88</td>
</tr>
<tr>
<td>Loss on ignition</td>
<td>2.1</td>
<td>0.9</td>
</tr>
</tbody>
</table>
3.2.2 Undensified Silica Fume

Undensified silica fume supplied by Elkem Company was used throughout the experimental process. The chemical composition is given below in Table 33.

Table 33 - Chemical composition of undensified silica fume

<table>
<thead>
<tr>
<th>Oxides</th>
<th>% by weight</th>
</tr>
</thead>
<tbody>
<tr>
<td>SiO₂</td>
<td>95.5</td>
</tr>
<tr>
<td>C</td>
<td>1</td>
</tr>
<tr>
<td>Fe₂O₃</td>
<td>0.3</td>
</tr>
<tr>
<td>Al₂O₃</td>
<td>0.7</td>
</tr>
<tr>
<td>CaO</td>
<td>0.4</td>
</tr>
<tr>
<td>MgO</td>
<td>0.5</td>
</tr>
<tr>
<td>K₂O</td>
<td>1.0</td>
</tr>
<tr>
<td>Na₂O</td>
<td>0.4</td>
</tr>
<tr>
<td>Loss on Ignition</td>
<td>2</td>
</tr>
<tr>
<td>Bulk Density</td>
<td>260 – 380 kg/m³</td>
</tr>
<tr>
<td>PH value</td>
<td>6.5-8.5</td>
</tr>
<tr>
<td>Specific gravity</td>
<td>2.20-2.30</td>
</tr>
<tr>
<td>Specific surface area</td>
<td>15 to 30 m²/g</td>
</tr>
</tbody>
</table>

3.2.3 High Range Water Reducing Agent

Ultra-high strength fiber reinforced concrete has very less water to cement ratio to achieve the higher strength by reducing the pore spaces. To obtain the good workable mix, Sika 2100 high range water reducer was used (Figure 58). It can achieve water reduction up to 45% at higher dosages. Sika Viscocrete 2100 meets the requirements of ASTM C-494 (57) Types A and F. Sika 2100 is suitable for making self-compacting concrete and improves the properties of fresh and hardened concrete. The properties of Sika Viscocrete 2100 are presented in Table 34.
Table 34 - Technical data sheet for Sika Viscocrete 2100

<table>
<thead>
<tr>
<th>Type</th>
<th>Property</th>
</tr>
</thead>
<tbody>
<tr>
<td>Appearance</td>
<td>Light blue liquid</td>
</tr>
<tr>
<td>Specific gravity</td>
<td>1.1</td>
</tr>
<tr>
<td>Basis</td>
<td>Polycarboxylate polymer technology</td>
</tr>
<tr>
<td>Toxicity</td>
<td>Non toxic</td>
</tr>
</tbody>
</table>

Figure 58 - Sika Viscocrete 2100

Apart from Sika Viscocrete 2100, Melflux 4930 F from BASF high performance superplasticizer was also used as shown in Figure 59. The properties of Melflux 4930 F are presented in Table 35.

Table 35 - Technical data sheet for Melflux 4930 F

<table>
<thead>
<tr>
<th>Physical shape</th>
<th>Powder</th>
</tr>
</thead>
<tbody>
<tr>
<td>Appearance</td>
<td>Yellowish to Brownish color</td>
</tr>
<tr>
<td>Drying loss, (%)</td>
<td>Max.2.0</td>
</tr>
<tr>
<td>Bulk density (kg/m3)</td>
<td>300 to 600</td>
</tr>
<tr>
<td>Dosage recommendation (1%)</td>
<td>0.05 to 1.0</td>
</tr>
</tbody>
</table>
3.2.4 Steel Fiber

Steel fibers (Figure 60) with aspect ratio 60 from Fibercon International Company were used in the experiments. The properties of the fibers are given below in the Table 36.

Steel fibers are used to improve the hardened concrete properties.

Table 36– Properties of steel fibers

<table>
<thead>
<tr>
<th>Property</th>
<th>Steel fibers</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length (mm)</td>
<td>13</td>
</tr>
<tr>
<td>Diameter (mm)</td>
<td>0.20</td>
</tr>
<tr>
<td>Density (gm/cm$^3$)</td>
<td>7.8 – 8.0</td>
</tr>
<tr>
<td>Tensile strength (MPA)</td>
<td>1000 (145 ksi)</td>
</tr>
</tbody>
</table>
3.2.5 Aggregates

Aggregate is relatively inexpensive material for making concrete. It is treated customarily as inert filler. The primary concern of aggregate in mix design for Ultra-High Performance concrete is gradation, maximum size and strength. Providing that concrete is workable, large size particles are undesirable for producing UHPC. The nominal size ranges from 0.15 to 0.6 mm (0.6 mm is the bigger size). It is very important to make sure that the sand is very clean, since a layer of clay and silt particles will reduce the cement aggregate bond strength, and also presence of clay and silt particles increase the demand for water. It is always better to ensure that the sand is very clean before concrete is being mixed. Figure 61 shows the difference between cleaned and un-cleaned sand.
The conventional river sand meeting the requirements of ASTM C33 (58) Gradation requirement was used in this study. The quartz sand is obtained from Tucker supply, Akron, OH. It has specific gravity of 2.65. Sand was cleaned using sieve no 200 and dried at 90°C for 24-48 hours to remove excessive moisture before being mixed with other ingredients of UHSFRC. Figure 62 shows the gradation curve for sand used.

Figure 61 - Difference showing between cleaned and un-cleaned sand

Figure 62 - Sieve analysis of sand from tucker supply
3.2.6 Water

Tap water supplied in the university was used in all concrete mixes as well for curing. The water is potable containing less impurities.

3.3 Mix Design

Various trails were conducted to get the optimum design mix. The standard mix design is used for all the experiments.

3.3.1 Mix Proportion

The details of the mix proportions are given in Table 37 and Table 38 with two different superplasticizers.

<table>
<thead>
<tr>
<th>Table 37 - Mix proportion of UHSFRC (Liquid HRWRA)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mix proportion by weight</td>
</tr>
<tr>
<td>--------------------------</td>
</tr>
<tr>
<td>Kg/m³</td>
</tr>
<tr>
<td>Lb/ft³</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Table 38 - Mix proportions of UHSFRC (Powdered HRWRA)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mix proportion by weight</td>
</tr>
<tr>
<td>--------------------------</td>
</tr>
<tr>
<td>Kg/m³</td>
</tr>
<tr>
<td>Lb/ft³</td>
</tr>
</tbody>
</table>
3.3.2 Mixing Procedure

After the selection of needed constituent materials, all materials were weighed properly. Mixing was done in Imer Mortarman 120 plus mixer (Figure 63), and ensuring that all the particles were properly mixed along with steel fibers. Mixing procedure is mentioned as follows

1) Adding 40% of the superplasticizer to water
2) Placing all the dry materials in the mixer (cement, silica fume, sand, fibers) and mixed for 3 minutes to ensure proper mixing
3) Adding water (with 40% superplasticizer) to the dry materials slowly for 2 minutes
4) Waiting for 1 minute and adding the remaining superplasticizer for 30 seconds.
5) Continued the mixing until UHSFRC changes from dry power to a thick paste.

Figure 63 - Imer 120 plus Mortarman mixer
After final mixing, the mixer was stopped and fresh homogenous concrete was transferred into the pan and then into the molds. The casting of all the UHSFRC specimens were done in 20 minutes after being mixed. All the specimens were covered with plastic to avoid the evaporation of water. Figure 64 shows the addition of steel fibers into the concrete mixer.

![Figure 64 - Addition of fibers to concrete](image)

3.3.3 Casting and De-molding

Necessary steps were taken to ensure the effective production of the specimens as the results depends on the output of the casted specimens. Before casting foam oil was applied to the cylinders and beams and other molds to release the friction and get the good surface at the sides and the bottom. UHSFRC is a self-compacting concrete and there was no need of vibration. The specimens were left covered with polythene for 24 hours and then de-molded and transferred to the curing room.
3.3.4 Different Curing Regimes

First curing regime followed was the moist curing (normal water curing). Mechanical properties such as compression, tension, flexural, rebar pull out test, reinforced concrete beams, shear keys, impact test were conducted using this curing regime.

In the second curing method specimens were de-molded after 24 hours and specimens were heat cured in water bath at 122°F (50°C) (Figure 65). Then, specimens were removed from water bath and dry cured at 392°F (200°C) for two days prior to testing. The mechanical properties of the concrete such as compression, tension and flexural strength tests were conducted by this curing regime.

Figure 65 - Samples kept in hot water chamber

3.4 Specimen Preparation and Testing Procedures

This section describes about the molds prepared and tests conducted according to the ASTM standards.
3.4.1 Slump flow and T500 test

It is very important to measure both the fresh and hardened concrete properties. Since ultra-high strength concrete has very less water to binder ratio, it is important to ensure its flowable properties by meeting the required standards.

The slump flow and T500 time test, measures the flowability and flow rate of self-compacting concrete in the absence of any obstructions. Usually two parameters are measured which are flow speed and flow time. The results is an indication of the filling ability of self-compacting concrete. T500 time test is a method to measure the speed of flow and hence viscosity of self-compacting concrete.

ASTM C1611 (59) standards were followed. The test method is considered applicable to self-consolidating concrete having maximum size not exceeding 25 mm (1”). The slump test is very similar to conventional concrete test and the cone can be either used the same way or inverted way.

As shown in the figure 66 the slump cone was inverted and filled till the top and extra portion was taken off. The slump cone was gently lifted and the timer was started to record till time until the concrete flows. Standard slump cone of size 4X8” top and bottom diameter and height of 12” was used for the test.

According to ACI 237R-07, common range of slump flow for SCC is between 18” to 30”. T500 time of generally 3 to 7 sec is acceptable for all civil engineering applications.
3.4.2 Compression Tests

Ultra-high strength fiber reinforced concrete is very well known for its compressive strength which effects the other mechanical properties of the concrete such as split tension, flexural tensile strength and bond. When the work started initially, cylinders of size (3”X6”) were used for measuring the compressive strength of the concrete. Due to the effect of uneven flatness results of cylinders completely scattered. Due to restrictions by ASTM, any concrete exceeding 12000 psi, neoprene pads cannot be used. However, rubber pads were used to check the behavior during the test. The concrete was totally stuck to the rubber pads producing unpleasant results as shown in Figure 67.

Figure 66 - Slump test
Figure 67 - Deterioration of testing pads due to high compressive strength

From the figure it is very clear that the pads were totally deteriorated and the concrete was stuck in them causing difficulty to take sample out. ASTM C1231 (60) states that un-bonded caps are not to be used for acceptance testing of concrete with compressive strength below 1500 psi (10 MPa) or above 12,000 psi (85 MPa).

After neoprene pads were completely destroyed, then tried using the ASTM C-167 (61) Sulfur based capping compound which attains 8000 psi strength in 2 hours to make the surface flat. ASTM C39 (62) Procedure was followed to perform the test. Interestingly, capping compound failed first then vertical cracks were observed in the cylinder. Figure 68, shows the failure of the capping compound.
Capping compounds are reliable only till 10,000 psi concrete. If the more vertical cracks are appearing in the cylinder then capping compound is not good or cannot be used. So it is better to grind both the top and bottom surfaces of the cylinder to ensure the full flatness. The machines for making the surface flat are expensive, and unavailable readily.

Considering all these problems more literature review was carried out to find out the effect of geometry on compressive strength and resulted in using the 2” cube specimens as shown in Figure 69, which does not need to be flattened further.
Hobart food mixer was used to make the cube specimens as the volume of the cubes is small and cannot mix the small quantity in the Imer 120 plus mixer. Dry ingredients were mixed well for 3 minutes, 40% of the superplasticizer was mixed with water, adding water to the materials for 2 minutes slowly, and waiting for a minute then for about 30 sec the remaining superplasticizer was added then continued mixing till the concrete became good workable paste. The concrete containing two types of cements (type 1&3) and two superplasticizers were cast at the same time.

ASTM C109 (63) dimensional tolerance Econ –O-cube molds were used for casting the cubes. Foam oil was applied on all the cubes and the grout was directly transferred from mixer to the molds with no vibration. Cube molds were covered with plastic sheets then de-molded after 24 hours and transferred into curing room.

Cubes were taken out from curing room and dried for one day and tested according to ASTM C109, loading rate of 82 psi/s was followed as shown in Figure 70, similarly the same procedure was followed for 28 days as well and load carrying capacity (lb) and strength (psi) were recorded.
3.4.3 Split tensile strength test

Split tensile test is the test to measure tensile capacity of the concrete indirectly. It measures the tensile strength of the concrete by compressing a cylinder through a line load applied along its length. Concrete is strong in compression and weak in tension. Direct tension test is difficult to perform that’s the reason split tension test was performed on the concrete. Cylinder specimens of 4”x8” size were casted for the following test. Cylindrical molds were coated with foam oil before transferring the grout into them. Imer 120 mortar mixer was used for mixing according to the mixing procedure discussed earlier. Concrete containing Type 1 and type 3 cements and two different superplasticizers were used for testing.

ASTM C496 (64) procedure was followed to perform the test and loading rate maintained to be 100-200 psi/min (Figure 71). The specimens were casted and allowed to
set for 24 hours. After 24 hours specimens were de-molded and kept in the moist curing room. After 28 days samples were taken out, dried and then tested.

![Figure 71 - Split tensile strength set up](image)

The most important thing is when applying the load, tensile strength is independent of sample size but it purely depends on the concrete quality. Each concrete specimen is laid in horizontal position, and the load was applied to create uniform tensile stress in cylinder. The splitting tensile strength equation is given by

$$f_t = \frac{2P}{\pi DL} \hspace{1cm} \text{(30)}$$

where

- $f_t$ = Split tensile strength, psi
- $P$ = Maximum applied load
- $D$ = diameter of the cylinder
- $L$ = length of the cylinder
3.4.4 Flexural strength test

Flexure strength is one of the measure of tensile strength of concrete. It is a measure of unreinforced concrete beam or slab to resist failure in bending. ASTM standard 4”X4”X14” specimens were used for testing. These specimens were casted with both Type I and Type III cements, along with different superplasticizers and curing procedures. The casting and testing procedure as per specification provided by ASTM C 78 (65) were followed. After casting the specimens, they are allowed to set for 24 hours. The specimens were removed from curing room and tested for 7 and 28 days. The corners were grounded to make the surface clean and top surface was made smooth to apply load. The beam is rested on the supports with a clear span of 12”. The testing was performed on universal testing machine with at an average loading rate of 30-50 lb/sec (Figure 72).

![Figure 72 - Testing on universal testing machine](image)

Flexural strength is calculated by

\[
\frac{M}{I} = \frac{f}{y} \quad \text{........................................... (31)}
\]

\(M = \text{Bending moment in lb-in} \)
\[ I = \text{Moment of inertia of the section in in}^4 \]
\[ f = \text{bending stress, psi} \]
\[ y = \text{distance from neutral axis, in} \]

3.4.5 Reinforced Concrete Beam Test

The main intension of this test is to study the performance of Ultra-High Strength Fiber Reinforced Concrete with steel reinforcement. Wood molds were prepared 28”x12”x3” as shown in the Figure 73. Reinforcing bars #4 (0.5” diameter) were placed at the bottom of the beam leaving clear cover of 0.5” and 2% steel fibers. The wood mold is first applied with form oil and beams were casted as shown in the figure below. No vibration was used as UHSFRC is self-compacting concrete. Slabs were tested at 28 days under three point bending according to ASTM C78 specifications.

![Figure 73 - Beam specimen molds](image.png)

After casting the beams they were kept for 24 hours to set, covered with plastic sheet. After 24 hours the beams were de-molded and transferred into curing room. Only moist curing practice was implemented to cure the samples. After 28 days of curing the
samples were taken out and dried for 24 hours before testing them. Figure 74, shows the pouring of concrete.

![Figure 74 - Concrete pouring directly from machine](image1)

From the above figure it is clearly seen that the concrete is self-compacting and needed no vibration, and no need of surface levelling as well.

Beams were de-molded after 24 hours and transferred them in the curing room Figure 75 shows the test setup for flexure.

![Figure 75 - Testing the beam](image2)
Beams were rested on the two supports leaving 2” on both the sides. So the clear span of the beam was 24” and loaded at the center. Dial gauges were kept at the center on both the sides to record the deflection for every 200 lb load increment. Loading rate of 30-50 lb/sec was maintained during the test. The Peak load was recorded along with the dial gauge readings.

Beam was analyzed as discussed in the literature chapter in accordance with ACI 544.4R-88. In addition to the steel reinforcement, the effect of the fibers are also included in the calculations.

3.4.6 Rebar pull out test

Bond is necessary not only to ensure adequate level of safety allowing composite action of steel and concrete, but also to control structural behavior along with sufficient ductility. The bond in reinforcing concrete members depends on many factors, includes size of the rebar, stress state in both concrete and steel.

Bond in reinforced concrete (RC) refers to the resistance of surrounding concrete against pulling out of reinforcing bars. If the bond resistance is inadequate, slipping of reinforcing bar occurs destroying the composite action. In reinforced concrete members sudden loss of bond between rebars and concrete causes brittle failure.

To check the rebar pull out strength, 6”x6”x6” cubes were made with an embedded length of 2” and 4” with two different super plasticizers. Figure 76 shows preparation of cube samples. Cube samples (6”x6”x6”) were made with wood and foam oil was applied making sure that it does not touch the reinforcing bars. A small size wooden sheet was placed at the bottom of the cubes so that the rebar does not slip out. After placing the rebar it was taped around.
Cylinders (6"X8") were also prepared to check the effect of size of the specimen as shown in Figure 77.
Cylinder samples were cast the same way as those of cube samples. The total length of the rebar was 30” and the required embedment length was inserted into the cylinder by making a small hole at the bottom and a square wooden sheet. Figures 78 and 79 show the filling of the samples with concrete.

Figure 78 - Filling cube samples for pull out test

Figure 79 - Filling of cylinders for pull out test
Cubes and cylinder were cast on the same day and left them to set for 24 hours covered them by plastic sheets. After 24 hours the samples were taken out and kept in the water buckets in such a way that water does not directly contact the rebar to avoid corrosion. After 28 days samples were taken out and dried for one day and then tested on universal testing machine by maintaining the loading rate of 30 lb/sec, (Figure 80). Trials were made to achieve a moderate loading rate and the same was maintained so that the testing time would be reduced.

![Figure 80- Test set up for both cylinder and cube samples](image)

The top part of the rebar was gripped firmly and the bottom part was placed with rubber and steel plate to ensure the flatness.

The Table 39, gives the information about the embedment lengths used along with the superplasticizers used.
Table 39 – List of the specimens

<table>
<thead>
<tr>
<th>Superplasticizer</th>
<th>Embedded length</th>
<th>No of specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sika 2100 Cube (6X6X6”)</td>
<td>2”</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>4”</td>
<td>2</td>
</tr>
<tr>
<td>Sika 2100 Cylinder (6X8”)</td>
<td>6”</td>
<td>2</td>
</tr>
<tr>
<td>Melflux (Cube 6X6X6”)</td>
<td>2”</td>
<td>2</td>
</tr>
<tr>
<td>Melflux 4930 Cylinder (6X8”)</td>
<td>6”</td>
<td>2</td>
</tr>
<tr>
<td>Melflux 4930 Cube (6X6X6”)</td>
<td>4”</td>
<td>2</td>
</tr>
</tbody>
</table>

3.4.7 Application of UHSC Grout in Shear Keys

Joint is the most crucial part in box beam girders. Adjacent box beam girders use grouted shear keys to transfer load between the beams.

The concept of using Ultra high strength fiber reinforced grouting material as a joint material between two precast concrete components is not new. Two projects which were completed in Denmark in 1995, used commercially available Ultra-High Performance Concrete as a closure fill material in the connection of slab elements in a building.

The widespread popularity of the adjacent pre-stressed box beam bridge can be attributed to number of factors

- Erection of these bridges are relatively quick and simple when compared to other bridge types.
- Minimal field labor is required
- Better quality control is maintained in a concrete casting plant when compared to pouring of the concrete in the field
- No field drilling or welding is required
The concern of using adjacent box beam girders is failure of shear keys. Box girders are set side by side on bearing pads and connected with transverse tie rods. Some states use transverse post tensioning and some states do not connect the girders. After the beams are placed, grout is poured into an octagonal shape gap between the beams which is referred as either shear key or keyway. When the grout hardens, beams are locked up together and acts as a monolithic bridge deck. In addition to providing the load transfer mechanism, shear keys also intend to seal the joints between the girders. It is noticed that the shear key tends to crack and leak even when a waterproofing membrane is in a place. This allows deicing chemicals to pass through the cracks and attack the concrete, and corrode the pre-stressed tendons and eventually break. It is believed that, failed shear keys can result in overloading the girder because the load is not distributed to adjacent girders.

The experimental work is as follows

The Ultra-High Strength Fiber Reinforced Concrete is well known for its mechanical properties, but to check its bonding ability with an existing concrete or new concrete, it is necessary to do a bond test. In order to test the bond for the grout, shear key was a preferred location. Small size box girders were prepared in the lab as shown in the Figure 81.

Initially no reinforcement, no tie rods were used in the box beams to test for keyways. The size of the specimens were scaled down and smaller samples were prepared as shown in the Figure 82.
1.5” spacing at the top and 3” spacing in the middle of the keyways were used according to ODOT specifications. The specimens were casted with 10,000 psi concrete and after 1 day setting, they were de-molded. 12 specimens were casted to check the different surface action.

Figure 82 - Picture showing dimensions of the shear key
After de-molding the samples, they were kept in place for filling with the grouting material. Different surfaces were used for the samples.

(i) Without any surface preparation
(ii) Sand blasting
(iii) Cement slurry
(iv) Sand Blasting + cement slurry

Firstly, no surface preparation has been done to the shear keys, the key was then grouted. Secondly, surface was prepared by sand blasting, (Figure 83) to make it rough so that the gout would adhere to the key surface very well. Cement slurry was used in 3:2 (cement: water) ratio for some of the specimens.

![Sand blasted surface and As cast concrete surface](image)

Figure 83 - Sand blasted and no surface preparation samples

Aluminum oxide sand blasting is used for roughening the surface of the key surface. 3 samples were sand blasted prior to filing, with the grouting material.

All the samples were made to fit between the wooden planks so that they cannot move and no spacing would cause the leakage of the grout shown in Figure 84.
Figure 84 - Prepared samples for grouting

Figure 85 shows the specimens were applied with cement slurry, 2 hours prior to the filing of the key with ultra-high strength grout.

Figure 85 - Cement slurry prepared samples

Materials were made ready and the same procedure of mixing was followed as discussed earlier and the grout after mixing was taken into the buckets and directly transferred into the keyways as shown in Figure 86. A slight vibration was done so that grout could not leave any spaces, more vibration leads to the sinking of the steel fibers.
After grouting, the samples were left out for 24 hours to get hardened. Then they were de-molded and transferred into curing room. All the samples were cured for 28 days, and were taken out of the curing room for testing as shown in the Figure 87.

As shown in the Figure 87, samples were wrapped with plastic so that there is no spalling of concrete after de-bonding. Dial gauges were placed on 6 sides to measure the deflection at every certain load interval. The loading rate of 30-50 lb/sec was maintained throughout the test Different surface prepared samples were tested on the same day and the average of 3 samples were taken.
3.4.8 Impact resistance test

Concrete has been used extensively as a protective structure to resist blast and impact loads for many years. Existing concrete structures designed without consideration of blast or impact can be vulnerable under unexpected extreme loads. Statically determined properties of concrete such as compression, tension, and flexure may not be used to predict the behavior of concrete subjected to high stress rates, those associated with impact, blast, and earthquake. To increase the impact resistance of concrete research has been done using different fibers and Fiber Reinforced Polymers (FRP). Since the conventional testing machines may not be used to generate such higher rates of loading, special test setup is required.

Slab panels of 20”X20”X3” were casted with conventional concrete and Ultra-High Strength Concrete with the same reinforcement ratio to evaluate the difference between both the concretes. The reinforcement ratio of 0.73% is used in either direction. The Figure 88 shows the reinforcement details of the slab.

Figure 88 - Reinforcing details of the slab
The Figure 89, shows the impact test set up. The test setup has a total weight of 300 lbs. of steel ball included with a cylindrical drum. This was dropped on the specimens from different heights, to check the impact energy absorption capacity of the slabs. The slabs were rested on the wooden frame to ensure support from all directions and the whole set up was resting on two heavy I beams which are tied together.
CHAPTER IV

RESULTS AND DISCUSSION

This chapter presents the results for various tests conducted on Ultra-High Strength Concrete grouting material. The results includes summary of fresh and mechanical properties such as slump test (T500 test), compressive strength, split tensile strength, flexural strength, reinforced concrete beams, rebar pullout strength test, application of grouting material in shear keys/key ways, and impact test.

4.1 Slump test

Standard 12” slump cone was used to measure the slump, 3 trails were performed to take the average of spread and time was noted. The requirements of spread and time for UHSFRC are shown in Table 40. The figure 90, shows the flow diameter of the grout.

Table 40 - Min and maximum requirements of slump flow and T500 time

<table>
<thead>
<tr>
<th>Test</th>
<th>Unit</th>
<th>Min</th>
<th>Max</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slump flow</td>
<td>mm (in)</td>
<td>550 (22)</td>
<td>850 (34)</td>
</tr>
<tr>
<td>T500 slump flow</td>
<td>Sec</td>
<td>2</td>
<td>9</td>
</tr>
</tbody>
</table>
From the Table 41, it is clear that UHSFRC has sufficient flow and meets the requirement criteria even with steel fiber content.

Table 41 – The Slump values recorded from experiments

<table>
<thead>
<tr>
<th>#</th>
<th>Slump flow (in)</th>
<th>T500 slump flow (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>25</td>
<td>4</td>
</tr>
<tr>
<td>2</td>
<td>26</td>
<td>4.5</td>
</tr>
<tr>
<td>3</td>
<td>25</td>
<td>4</td>
</tr>
</tbody>
</table>

4.2 Compressive Strength

The compressive strength of different curing practices and different cements were evaluated. 7 and 28 days compressive strengths have been recorded and tabulated in Tables 42 and 43. As discussed in the earlier chapter two different curing methods were used moist curing (MC) and heat curing (HC). In the moist cured process cube samples were de-molded after 24 hours and put them in the moist curing room. In the heat curing practice, the cubes were de-molded after 24 hours and kept them in the water bath at 122°F (50°C)
and then the specimens were removed from water bath and kept them in the oven at 392°F (200°C). Samples were taken from curing rooms and tested according to ASTM C109 standards. All the cube samples maintained the same rate of loading. Unit weight of the cubes were found out to be between 145-150 lb/ft³.

Table 42 – Compressive at 7 days test results

<table>
<thead>
<tr>
<th></th>
<th>Type of curing</th>
<th>Failure load(lb)</th>
<th>Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type 1 cement + Sika 2100</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>MC</td>
<td>51832</td>
<td>12958</td>
</tr>
<tr>
<td>2</td>
<td>MC</td>
<td>52232</td>
<td>13058</td>
</tr>
<tr>
<td>3</td>
<td>MC</td>
<td>53168</td>
<td>13292</td>
</tr>
<tr>
<td>Type 1 cement + Sika 2100</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>HC</td>
<td>104528</td>
<td>26132</td>
</tr>
<tr>
<td>2</td>
<td>HC</td>
<td>106420</td>
<td>26605</td>
</tr>
<tr>
<td>3</td>
<td>HC</td>
<td>106980</td>
<td>26745</td>
</tr>
<tr>
<td>Type 3 cement + Sika 2100</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>1</td>
<td>MC</td>
<td>64748</td>
<td>16187</td>
</tr>
<tr>
<td>2</td>
<td>MC</td>
<td>65400</td>
<td>16350</td>
</tr>
<tr>
<td>3</td>
<td>MC</td>
<td>66608</td>
<td>16652</td>
</tr>
<tr>
<td>Type 3 cement + Melflux 4930</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>MC</td>
<td>63608</td>
<td>15902</td>
</tr>
<tr>
<td>2</td>
<td>MC</td>
<td>64700</td>
<td>16175</td>
</tr>
<tr>
<td>3</td>
<td>MC</td>
<td>64968</td>
<td>16242</td>
</tr>
</tbody>
</table>
### Table 43 – Compressive strength at 28 days test results

<table>
<thead>
<tr>
<th></th>
<th>Type of curing</th>
<th>Failure load(lb)</th>
<th>Strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Type 1 cement + Sika 2100</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>#</td>
<td>Type of curing</td>
<td>Failure load(lb)</td>
<td>Strength (psi)</td>
</tr>
<tr>
<td>1</td>
<td>MC</td>
<td>66368</td>
<td>16592</td>
</tr>
<tr>
<td>2</td>
<td>MC</td>
<td>66808</td>
<td>16702</td>
</tr>
<tr>
<td>3</td>
<td>MC</td>
<td>68608</td>
<td>17152</td>
</tr>
<tr>
<td><strong>Type 1 cement + Sika 2100</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>#</td>
<td>Type of curing</td>
<td>Failure load(lb)</td>
<td>Strength (psi)</td>
</tr>
<tr>
<td>1</td>
<td>HC</td>
<td>111368</td>
<td>27842</td>
</tr>
<tr>
<td>2</td>
<td>HC</td>
<td>112700</td>
<td>28175</td>
</tr>
<tr>
<td>3</td>
<td>HC</td>
<td>113600</td>
<td>28400</td>
</tr>
<tr>
<td><strong>Type 3 cement + Sika 2100</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>#</td>
<td>Type of curing</td>
<td>Failure load(lb)</td>
<td>Strength (psi)</td>
</tr>
<tr>
<td>1</td>
<td>MC</td>
<td>84060</td>
<td>21015</td>
</tr>
<tr>
<td>2</td>
<td>MC</td>
<td>84308</td>
<td>21077</td>
</tr>
<tr>
<td>3</td>
<td>MC</td>
<td>88308</td>
<td>22041</td>
</tr>
<tr>
<td><strong>Type 3 cement + Melflux 4930</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>#</td>
<td>Type of curing</td>
<td>Failure load(lb)</td>
<td>Strength (psi)</td>
</tr>
<tr>
<td>1</td>
<td>MC</td>
<td>75800</td>
<td>18950</td>
</tr>
<tr>
<td>2</td>
<td>MC</td>
<td>76668</td>
<td>19167</td>
</tr>
<tr>
<td>3</td>
<td>MC</td>
<td>78920</td>
<td>19730</td>
</tr>
</tbody>
</table>
Figures 91 and 92 show the failure pattern of the moist cured and heat cured samples respectively. From the moist cured samples just minor cracks were observed, whereas heat cured samples failed catastrophically. Figures 93 and 94, give the graphical representation of 7 days and 28 days compressive strength of all the specimens.

Figure 91 - Failures of the moist cured cubes

Figure 92 - Failure of the heat cured sample
Discussion:

From the Table 44, it can be observed that the compressive strength of heat cured samples are very high. Due to lower water cement ratio in ultra-high strength concrete full hydration of cement and silica fume are never achieved. The pozzolanic reaction of silica
fume greatly depends on the temperature of curing, heat curing has the potential to accelerate the pozzolanic reaction.

Type 1 moist cured concrete samples have lesser compression strength values when compared to type 3 cement moist cured concrete samples. In fact the 7 days compressive strength results of concrete specimens with type 3 cement, were equal to 28 days compressive strength of concrete samples with type 1 cement.

The 7 days strengths of concrete specimens with type 3 cement and different superplasticizers have very close compression strength of 16400 psi, whereas 28 days compressive strength of concrete specimens with Melflux 4930 observed to be less when compared to Sika 2100.

The 7 and 28 days results of Heat cured samples were very high and more tests have been performed to evaluate other mechanical properties. Heat curing might be artificially inflating the strength of the concrete, flexural tensile strength and split tensile strength were needed to be performed to understand it better.

The 7 days heat cured samples have 102% more compressive strength when compared to moist cured samples. 28 days strength of heat cured samples made of type 1 cement was 31% higher than type 3 moist cured.

The use of type 3 cement has resulted in higher early strength of the grout. The 7 days strength of specimens made of type 3 cement is 25% higher than specimens made of type 1 cement and 28% higher in 28 days. This concludes that type 3 cement can be used, wherever there is a need for high early strength (i.e. in shear keys).
4.3 Splitting Tensile Strength

Split tensile test was performed on 4”X8” Cylinders. The cracking pattern on some of the cylinders was observed to be abnormal. So to check the difference, 4”X4” cut cylinders were tested following the ASTM C496 specifications. The results of 3 samples were recorded and tabulated in Table 44. Figure 95, shows the graphical representation of split tensile strength of specimens.

Table 44 – Splitting tensile strength test results

<table>
<thead>
<tr>
<th>Curing Method</th>
<th>Specimen 1 (psi)</th>
<th>Specimen 2 (psi)</th>
<th>Specimen 3 (psi)</th>
<th>Average (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type 1 + Sika 2100 MC</td>
<td>1522.5</td>
<td>1696.5</td>
<td>1740</td>
<td>1653</td>
</tr>
<tr>
<td>Type 1 + Sika 2100 HC</td>
<td>1812.5</td>
<td>1899.5</td>
<td>1870.5</td>
<td>1860</td>
</tr>
<tr>
<td>Type 3 + Sika 2100 MC</td>
<td>2392.5</td>
<td>2247.5</td>
<td>2305.5</td>
<td>2320</td>
</tr>
<tr>
<td>Type 3 + Mlelfux 4930 MC</td>
<td>1899.5</td>
<td>2102.5</td>
<td>2175</td>
<td>2059</td>
</tr>
</tbody>
</table>

Figure 95 – Graphical representation of Splitting tensile strength results
From the Table 44, it can be observed that type 1 cement moist cured samples have less split tensile strength compared to rest of the mixes. The Cylinders did not fail completely because, the fibers were holding the concrete intact. Figure 96, shows the failure pattern of the cylinders.

![Figure 96](image)

Figure 96 – Failure pattern of cylinders

Failure pattern of moist cured samples were in the plane of loading, while heat cured samples observed to fail out of the plane. The failed specimens are separated along the crack to observe the failure surface. It was very tough to break the samples as the fibers were holding firmly. The distribution of fibers were uniform all along the longitudinal section (Figure 97). Some of the fibers were observed to be oriented vertically whereas some of them were inclined, but most of the fibers were parallel.

Split tensile strength of moist cured and heat cured samples containing type 1 cement are more or less the same. It can be seen that due to use of type 3 cement there is an increase of 40% of split tensile strength at 28 days.
4.4 Flexural Strength

Flexural strength test was performed on 4”X4”X14” un-reinforced beams. Tests have been performed at 7 and 28 days.

Table 45, shows the flexural strength of the 3 beams at 7 days, for all 4 different design mixes. From the table it is clear that the flexural strength of type 3 cement is higher for 7 days, which is 3323.7 psi. It is very strange to observe that the flexural strength of heat cured samples is very less when compared to moist cured samples. Heat cured samples had the compressive strength of 26500 psi at 7 days whereas the compressive strength of moist cured samples was 13100 psi. Type 3 cement has the 7 days compressive strength of 16400 psi and has the higher flexural strength at 7 days. Table 46 shows the flexural strength of 3 samples for two different cements and superplasticizers used concrete at 28 days.
Table 45 - Flexural strength results at 7 days

<table>
<thead>
<tr>
<th>Curing Method</th>
<th>Specimen 1 (psi)</th>
<th>Specimen 2 (psi)</th>
<th>Specimen 3 (psi)</th>
<th>Average (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type 1 +Sika 2100</td>
<td>2295</td>
<td>2454</td>
<td>2483</td>
<td>2411</td>
</tr>
<tr>
<td>Type 1 +Sika 2100</td>
<td>2025</td>
<td>2399</td>
<td>2609</td>
<td>2344</td>
</tr>
<tr>
<td>Type 3+ Sika 2100</td>
<td>3299</td>
<td>3354</td>
<td>3052</td>
<td>3323.7</td>
</tr>
<tr>
<td>Type3+Mlelfux4930</td>
<td>2808</td>
<td>2966</td>
<td>3052</td>
<td>2942</td>
</tr>
</tbody>
</table>

Table 46 - Flexural strength results at 28 days

<table>
<thead>
<tr>
<th>Curing Method</th>
<th>Specimen 1 (psi)</th>
<th>Specimen 2 (psi)</th>
<th>Specimen 3 (psi)</th>
<th>Average (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type 1 +Sika 2100</td>
<td>2609</td>
<td>2723</td>
<td>2727</td>
<td>2686.2</td>
</tr>
<tr>
<td>Type 1 +Sika 2100</td>
<td>2402</td>
<td>2749</td>
<td>2743</td>
<td>2631</td>
</tr>
<tr>
<td>Type 3+ Sika 2100</td>
<td>3111</td>
<td>3375</td>
<td>3516</td>
<td>3334</td>
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<tr>
<td>Type3+Mlelfux4930</td>
<td>2959</td>
<td>3074</td>
<td>3319</td>
<td>3117</td>
</tr>
</tbody>
</table>

The 7 days flexural strength of heat cured samples has an average flexural strength of 2344 psi, whereas moist cured samples has 2411 psi. Due to the use of type 3 cement at 7 days there is 38% increase in flexural strength.

The failure pattern of all the moist cured samples observed to be similar and has more or less the same crack widths, but the heat cured samples have the crack width of 0.1” (Figure 100). Figure 98 and Figure 99 shows the graphical representation of the 7 and 28 days flexural strengths.
Figure 98 – Graphical representation of 7 days flexural strength

Figure 99 – Graphical representation of 28 days flexural strength

Figure 100 and 101, shows the widened cracks patterns of heat cured samples. All the beam samples failed the same way and fibers were not able hold the concrete.
All the 7 days and 28 days cured samples had the same failure pattern of cracks. While performing the test of heat cured samples, cracks appeared all of a sudden and the fibers were not able to hold the concrete, then the failure of the beam occurred.

Moist cured beams were tested for 7 and 28 days. After the beams were taken out from the curing room they were kept in dry conditions for 24 hours and tested. The cracking pattern of the moist cured type 1 or type 3 cement specimens were same. No sudden failure
was observed and crack widths were much lesser compared to heat cured samples as shown in Figure 102.

![Figure 102 - Difference in crack widths between moist cured and heat cured samples](image)

Increase in percentage of flexural strength from 7 to 28 days is not so high for type 1 and type 3 cements. Type 3 cement with Melflux 4930 grout has an increase in 6% of flexural strength at 28 days. Type 1 cement moist cured samples have increased flexural strength by 11.5% at 28 days. Heat cured samples have the lesser flexural strength at 7 days when compared to moist cured samples. Type 3 cement with Sika 2100, the flexural strength at 7 and 28 days was found to be equal with no increase in strength.

It is clear evident from the 3 tests that, the heat curing process artificially inflates the compressive strength, but the split tensile strength and flexural strength results are found be less compared to moist cured samples. (50°C+200°C) temperature has the less effect on split tension and flexural strength, compared to the compressive strength.
Beams were broken down after the test to check the orientation of fibers. Beams were very tough to break since the fibers were firmly holding the concrete. It was observed that distribution of fibers was even as shown in Figure 103 and Figure 104.

![Figure 103 - Distribution of fibers](image1)

![Figure 104 - Beam after breaking](image2)

On looking at the mechanical properties of the concrete and due to constraint of the materials it was decided to proceed further with moist curing practice.
4.5 Flexural behavior reinforced concrete fibrous concrete

The main intention of this test was to see the performance of Ultra-High Strength Grout material bonding with the reinforcing bars. The test has been carried out by using the concrete with type 1 cement. Along with the specimens made of Ultra-High Strength Fiber Reinforced Concrete, specimens made of conventional concrete with same reinforcement ratio were also tested. Table 47, gives the max load carrying capacity of UHSFRC beam specimens.

Table 47 – Max load for reinforced concrete beams of UHSFRC

<table>
<thead>
<tr>
<th>Specimen No</th>
<th>Max Load (Lb)</th>
<th>Type of Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>17252</td>
<td>Flexure</td>
</tr>
<tr>
<td>2</td>
<td>18052</td>
<td>Flexure</td>
</tr>
<tr>
<td>3</td>
<td>18551</td>
<td>Flexure</td>
</tr>
</tbody>
</table>

The average of 3 samples were taken to find out the flexural capacity of the beams made of UHSFRC and were with that estimated from ACI 544 method. Dial gauges were placed at the mid-span on either sides to measure the deflection. Load vs deflection graphs were drawn. Table 48, gives the max load carrying capacity of beam specimens made of conventional concrete.

Table 48 - Max load for reinforced concrete beams of Conventional concrete

<table>
<thead>
<tr>
<th>Specimen No</th>
<th>Max Load (Lb)</th>
<th>Type of Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>9105</td>
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<td>2</td>
<td>9200</td>
<td>Flexure</td>
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<tr>
<td>3</td>
<td>9240</td>
<td>Flexure</td>
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</tbody>
</table>
In conventional concrete specimens, the transfer of load from concrete to rebar which depends on the development of bond at the concrete rebar interface, resulting in more load carrying capacity. The Figure 103 shows the cracking pattern of the UHSFRC concrete specimen. It was observed that, only flexural cracks at the middle of the beam were developed. The crack propagation was limited at the center and did not widen up due to presence of steel fibers.

![Figure 105 – Cracking of reinforced concrete beams](image)

Tests were also done with conventional concrete maintaining the same rate of loading, and was noted that the load carrying capacity of the conventional concrete is half of that with ultra-high strength concrete specimens. i.e. the moment capacity of the fiber reinforced concrete is twice than the conventional concrete, due to the denser microstructure, absence of coarse aggregates, and cement silica reaction. The lower water cement ratio was very important parameter in Ultra-High Strength Concrete. Figure 106, shows the cracking pattern of UHSC beams.
Figure 106 – Failure pattern of the UHSFRC beams

Conventional concrete beams have different failure patterns than UHSFRC, which is not exactly the flexural failure, and this may be due to the improper bond of rebar with concrete, and may be due to absence of fibers. Figure 107 shows the cracking pattern of conventional concrete beams.

Figure 107 - Failure pattern of conventional concrete beam

The graph in Figure 108 shows the load deflection curve for conventional concrete and Ultra High Strength Concrete. At the same deflection, the load carried by Ultra High
Strength Fiber Reinforced Concrete is twice as compared to conventional concrete beam. The ultimate load carried by the beams with UHSFRC is also twice as much as conventional concrete beams. Moment carrying capacity of the beam calculations are done using ACI 544 standards.

![Deflection vs Load](image)

**Figure 108 - Load deflection graph for conventional and UHSFRC**

At the ultimate load, the max deflection observed in conventional concrete specimens was 0.23” so as in UHSFRC beam.

4.6 Rebar pull out strength test

#4 rebar (0.5” diameter) with yield strength of 60,000 psi (60 ksi) was used for testing all the specimens. 6” Cube and 6”X8” Cylinders were used for testing with different embedment lengths. Regardless of the type of the cement and type of the superplasticizer used, the stress in the rebar during the testing of every specimen exceeded the minimum specific yield stress of the steel.
Two types of superplasticizers and two types of cements were used for the test specimens.

Short form of the test specimens are as follows

**T1-1-C2-S**, represents Type 1 cement sample 1 using Cube with embedment length of 2” and Sika 2100 was used as a high range water reducer.

**T1-1-CY6-S**, represents Type 1 cement sample 1 using Cylinder with embedment length of 6” and Sika 2100 was used as a high range water reducer.

The same naming was followed for type 3 cement samples with Melflux 4930 superplasticizer. Figure 109, shows specimens for rebar pullout testing.

![Figure 109 - Specimens for rebar pullout testing](image)

Table 49 – Max tension force and failure mode

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Max Tension Force</th>
<th>Max Tension stress</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>lb (kip)</td>
<td>ksi (Mpa)</td>
<td></td>
</tr>
<tr>
<td>T1-1-C2-S</td>
<td>12678 (12.6)</td>
<td>64.6(445.5)</td>
<td>Splitting cracks radiated out from rebar</td>
</tr>
<tr>
<td>T1-2-C2-S</td>
<td>13829 (13.9)</td>
<td>70.9(489.0)</td>
<td>Splitting cracks radiated out from rebar</td>
</tr>
<tr>
<td>T1-1-C4-S</td>
<td>18124(18.1)</td>
<td>92.3(636.5)</td>
<td>Rebar fracture</td>
</tr>
<tr>
<td>Specimen</td>
<td>Max Tension Force</td>
<td>Max Tension stress</td>
<td>Failure mode</td>
</tr>
<tr>
<td>------------</td>
<td>-------------------</td>
<td>--------------------</td>
<td>-------------------------------------</td>
</tr>
<tr>
<td>T1-2-C4-S</td>
<td>19769 (19.7)</td>
<td>100.5 (693.1)</td>
<td>Rebar fracture</td>
</tr>
<tr>
<td>T1-1-CY6-S</td>
<td>21783 (21.78)</td>
<td>111.2 (766.9)</td>
<td>Rebar fracture</td>
</tr>
<tr>
<td>T1-2-CY6-S</td>
<td>19728 (19.7)</td>
<td>100.5 (693.1)</td>
<td>Rebar fracture</td>
</tr>
<tr>
<td>T3-1-C2-M</td>
<td>17277 (17.2)</td>
<td>87.7 (604.8)</td>
<td>Splitting cracks radiated out from rebar</td>
</tr>
<tr>
<td>T3-2-C2-M</td>
<td>18124 (18.1)</td>
<td>92.3 (636.5)</td>
<td>Splitting cracks radiated out from rebar</td>
</tr>
<tr>
<td>T3-1-C4-M</td>
<td>20395 (20.4)</td>
<td>104.0 (717)</td>
<td>Rebar fracture</td>
</tr>
<tr>
<td>T3-2-C4-M</td>
<td>21452 (21.5)</td>
<td>109.7 (756.5)</td>
<td>Rebar fracture</td>
</tr>
<tr>
<td>T3-1-CY6-M</td>
<td>23925 (24)</td>
<td>122.4 (844.13)</td>
<td>Rebar fracture</td>
</tr>
<tr>
<td>T3-2-CY6-M</td>
<td>23264 (23.3)</td>
<td>118.8 (819.3)</td>
<td>Rebar fracture</td>
</tr>
</tbody>
</table>

Discussion:

The yield strength of the rebar is 60 ksi. All the specimens surpassed the yield strength of the rebar. Specimen with Type 1 cement and 2” embedment length had 68 ksi tension stress and the failure was due to de-bonding i.e., splitting cracks from rebar.

Type 3 cement used grouting material with 2” embedment length has the similar failure of splitting cracks but the tension stress is 90 ksi, which is 33% increase due to the use of type 3 cement. Figure 110 shows the graphical representation of tension stress of a 2” embedment length specimens.

![Graphical representation of tension stress](image)

Figure 110 – Graphical representation of 2” embedment length tension stress
The Figure 111, shows the variation of tension stress of a 4” embedded cube specimens due to use of two different types of cements.

Figure 111 – Graphical representation of 4” embedment length tension stress

Figure 112, shows the variation in tension stress of a 6” embedded length specimens of type 1 and type 3 cement concretes.

Figure 112 – Graphical representation of 6” embedment length tension stress
Figure 113, shows the development of radial crack pattern of a 2” embedded length cube specimens.

![Cracks radiated out from rebar](image1)

Figure 113 - Cracks radiated out from rebar

4” embedment length of specimens made of two cements were also tested under similar conditions. The failure pattern was observed to be a rebar fracture (Figure 114). Due to use of type 3 cement, only 10% increase in tension stress was noted.

![Rebar fracture failure of 4” embedment length cubes](image2)

Figure 114 – Rebar fracture failure of 4” embedment length cubes
For 6” embedment length specimen’s cylinders were prepared. Cylinder casted with both the cement were tested. Rebar fracture failure was observed as shown in the Figure 115 and 116.

Figure 115- Rebar fracture failure of the 6” embedment length cylinders

Figure 116 - Rebar fracture after test
4.7 Shear key test

After studying the mechanical properties of the concrete, it was clear that we can produce ultra-high strength grout using naturally available materials rather than using expensive products, which are not readily available. Mechanical properties of the UHSFRC concrete gave a clear idea about its performance and it was decided to use for connecting the small box beam specimens with different prepared surfaces as listed in Table 50.

Table 50 – Different surface prepared list

<table>
<thead>
<tr>
<th>No of samples</th>
<th>Cement Slurry</th>
<th>Sand Blast</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>3</td>
<td>✓</td>
<td>X</td>
</tr>
<tr>
<td>3</td>
<td>X</td>
<td>✓</td>
</tr>
<tr>
<td>3</td>
<td>✓</td>
<td>✓</td>
</tr>
</tbody>
</table>

Where X = Surface not prepared using that specified method

✓ = surface prepared using that specified method

All the tests have been performed with the moist cured samples. No heat curing or pressure curing was applied. Samples were kept in the moist curing room for 28 days.

In all the cases of testing, the grouting material did not crack or fail, as the mechanical properties were good, but de-bonding was observed. Without any surface preparation samples have de-bonded at very less cracking load. Surface prepared with cement slurry and no sand blasting has lesser cracking load in all the cases. It was noticed that cement slurry reduced the strength drastically.

Cement slurry and sand blasted specimens have the 2nd largest load carrying capacity when compared to all the specimens. The failure was observed to be de-bonding and diagonal shear in the middle concrete unit.
4th specimen was only sand blasted. This specimen has the highest cracking load. Failure was in the middle of the concrete unit, but not at the interface of the grout and concrete. Limited by the strength of the concrete units but not the strength of the grouting material. Table 51, presents the average cracking load of the test specimens.

Table 51 – Results of the shear key test

<table>
<thead>
<tr>
<th>No</th>
<th>Specimen</th>
<th>Average Cracking Load (Lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>No cement no Sand Blast</td>
<td>8100</td>
</tr>
<tr>
<td>2</td>
<td>Cement Slurry no Sand Blast</td>
<td>5300</td>
</tr>
<tr>
<td>3</td>
<td>Cement Slurry and Sand Blast</td>
<td>24067</td>
</tr>
<tr>
<td>4</td>
<td>Only Sand Blast</td>
<td>34700</td>
</tr>
</tbody>
</table>

No cement no sand blast specimen, the failure mode is shown in the Figure 117. It was failed at the interface of beam and the grout, i.e. pure de-bonding. Grouting material did not fail or crack.

Figure 117 – Failure by de-bonding (no slurry no sand blast)
Figure 118, shows the failure pattern of specimens, surfaced with cement slurry and no sand blasting. It was observed that, the cement slurry was mixed with the grout material and did not adhere to the concrete surface.

Figure 118- Failure by de-bonding (cement slurry no sand blast)

Figure 119, shows the failure pattern of diagonal shear failure on the concrete specimens surfaced with sand blasting and cement slurry. It was concluded that the strength of the concrete unit is lesser than the strength of the applied grout.

Figure 119 – De-bonding and diagonal shear in the middle of the Concrete unit (slurry +sand blast)
Figure 120 shows the failure pattern of specimens surfaced with only sand blasting. These specimens carried the highest cracking load and resulted in a compression failure of the concrete unit.

![Image of concrete specimens showing failure pattern](image)

**Figure 120 – Failure of in the middle of the concrete unit (sand blasted)**

From no surface prepared specimens to cement slurry prepared specimens, load is decreased by 35%, as the cement slurry is mixed with the grout which weakens the bond.

From no surface preparation specimens to cement slurry and sand blasted specimens, there is an increase of 197% in loading capacity.

From no surface preparation specimens to only sand blasted specimens, there is an increase of 330% in loading capacity. The results are graphically shown in Figure 121.

![Graphical representation of average cracking load](image)

**Figure 121 – Graphical representation of average cracking load**
4.8 Impact test

Impact test gives the idea of how the material is performing to dynamic loads. So far the mechanical properties of concrete and bonding properties have been studied.

Initially 4 ft. height of drop was considered for the conventional concrete slabs, and UHSFRC slabs, to evaluate the difference between cracking pattern and impact energy absorption.

The Figure 122, shows the cracking pattern of conventional concrete panel, when dropped from a 4ft height. The measured impact energy was 1.68 KJ. Concrete was completely spalled out, it is clear that the energy absorption capacity of conventional concrete slabs is lesser than 1.68 KJ. On the other hand UHSFRC specimens were able to withstand 1.68 KJ, with minor cracks appearing at the bottom of the slab, no cracks were seen in the compression region of the slab.

![Figure 122 – Failure of the conventional slab specimens (4ft height drop)](image)

In UHSFRC only some minor cracks of 0.025” were observed as shown in Figure 123. As UHSFRC contained steel fibers in them, there was no spalling of concrete
observed. When the ball was dropped on the slabs, it was bounced twice indicating that it is hard enough to withstand the applied energy.

![Image](image1.png)

Figure 123 – Failure of the UHSFRC slab specimens (4ft height drop)

From 4ft height, both the conventional concrete slabs were failed. So it was decided to proceed further and 8ft height was selected for UHSFRC specimen. Cracks of 0.06” wide were measured at the bottom of the slab. The impact energy was theoretically found to be 3.4 KJ, and concrete is able to withstand.

![Image](image2.png)

Figure 124 – Failure of the UHSFRC slab specimens (8ft height drop)

Table 52, presents the impact energy absorption capacities of the slab panels.
<table>
<thead>
<tr>
<th>No of samples</th>
<th>Concrete</th>
<th>Drop Height (ft)</th>
<th>Impact Energy</th>
<th>Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>Conventional Concrete</td>
<td>4</td>
<td>1.68</td>
<td>Failed</td>
</tr>
<tr>
<td>1</td>
<td>UHSFRC</td>
<td>4</td>
<td>1.68</td>
<td>0.025” cracks width</td>
</tr>
<tr>
<td>1</td>
<td>UHSFRC</td>
<td>8</td>
<td>3.4</td>
<td>0.06” cracks width</td>
</tr>
</tbody>
</table>
CHAPTER V

CONCLUSIONS

5.1 Conclusion

After conducting extensive experimental research work, following conclusions can be drawn from the results.

- Ultra-High Strength Fiber Reinforced Concrete can be made by using materials available locally, with water binder ratio of 0.2 and 15% silica fume.
- From the mechanical properties such as compressive strength, tensile strength, flexural strength of concrete specimens with two different types of cements two high range water reducing agents, it can be concluded that type 3 cement concrete has properties higher than type 1 cement by using moist cured practice.
- Concrete with Type 1 cement has compressive strength of 13,100 psi and concrete with type 3 cement has compressive strength of 16,400 psi in 7 days. For 28 days, concrete with type 1 cement has a strength of 16,800 psi whereas concrete with type 3 cement has 21,500 psi.
- The highest compressive strength was achieved by using the heat curing practice, i.e., 26,500 psi in 7 days and 28,150 psi in 28 days.
• Split tensile strength of concrete with type 3 cement was 2,320 psi by using moist curing practice.

• Flexural strength of 3,330 psi was gained by using type 3 cement. The heat cured samples have lesser flexural strength than moist cured samples.

• It was observed that 50°C+200°C temperature is artificially inflating the compressive strength. Whereas the flexural strength, split tensile strength of heat cured samples are less effected by temperature.

• Reinforced concrete beam using both conventional concrete and UHSFRC, has been tested and found that the ultimate moment carrying capacity of UHSFRC is 100% higher than conventional concrete with same reinforcement ratio and similar cement type.

• In rebar pullout strength test, it was observed that, 2” embedment length was not sufficient to develop well bond with concrete hence resulted in radial cracks at the concrete steel interface. For 4” and 6” embedment lengths the rebar has fractured, and it is clear that concrete is stronger than rebar with well bond.

• For shear key test, the sand blasted specimens has the highest average cracking load among all the surface prepared samples. The load is equal to the HL-93 single axle load.

• The impact resistance of the UHSFRC specimens is 100% higher than the conventional concrete specimens and the failure pattern of UHSFRC specimens is not catastrophic.
5.2 Recommendations

This research study revealed significant potential for use of UHSFRC as a grouting material for shear key which can be more susceptible to corrosion problem. The higher strength can also be achieved by not using the materials which are really expensive.

- Durability properties of the concrete such as Rapid chloride ion penetration, Drying Shrinkage, Autogenous Shrinkage needs to be investigated.
- Effect of different types of the fibers like basalt, polypropylene, copper coated steel fibers, and hooked end steel fibers in different dosages has to be studied.
- The effect of different heat curing regime, such as 90° C+90° C has to be studied as the 50°C+200°C is just increasing the compressive strength of the concrete.
- Studies has to be done by increasing the silica fume dosage and decreasing the cement content and further reducing the water to cement ratio.
- Effect of different high range water reducing agents which are commercially available in the market has to be taken into account, with different dosages.
- Rebar pull out strength with different embedment lengths should be investigated.
- Beach sand which has the size lesser than 600 micron may be studied which would reduce the effort of sieving.
- Freeze-thaw cycle testing and coefficient of thermal expansion tests can be investigated.
- Different mineral admixtures such as usage of Fly Ash, GGBS in different proportions can be studied.
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<thead>
<tr>
<th>No.</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
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<td>23</td>
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</tr>
<tr>
<td>24</td>
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</tr>
<tr>
<td>26</td>
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</tr>
<tr>
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</tr>
</tbody>
</table>


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APPENDIX

FLOW CHART PRESENTATION OF EXPERIMENTS
Trial Mixes

Initially different mixers were used by maintaining the same water to binder ratio. Concrete still contained lumps on mixing for 45 minutes by conventional concreter mixer. The lumps in concrete can be avoided by having proper inclination of blades like as in high shear mixers.

Fine aggregates

The fine sand supplied from local supplier was used to study the effect of clean and uncleaned sand on properties of concrete.

Cement

The Type 1 cement used had lot of lumps and to break the lumps it had to be sieved and quantity passing through #200 sieve was used for initial work

Silica fume

Densified silica fume were used during the preliminary work. In general higher the carbon content, the darker is the silica fume. The carbon content of silica fume is affected by many factors relating to the manufacturing process, such as: use of wood chips versus coal, wood chip composition, furnace temperature, furnace exhaust temperature, and the type of product (metal alloy) being produced. Densified silica fume particles do not break when mixing in mortar, unlike the concrete where the mixing is more rigorous.
Superplasticizer

Sikament 686 Superplasticizer from Sika Corporation is used at an early stage of the work. 3% by weight of cementitious material is used and maintained constant for all the proportions. The mix derived was not workable needed lot of vibration.

![Sikament 686 Superplasticizer](image)

Figure 125 – Sikament 686 Superplasticizer

Mixer:

High shear mixer was unavailable at early stage of the work. Conventional mixers were used by adding the blades to them. But the grout at lower water cement ratio did not mix. It remained as powder even after mixing for 30 minutes.

Concrete mixer by Global Gilson Company was used for mixing 1ft³ quantity, cement silica fume, and sand were well mixed for 5 minutes, half of the high range superplasticizer was added to water and later water was slowly added for 2 minutes. After adding the water remaining superplasticizer was added and mixed for 30 minutes, all the ingredients were still in the powdered condition and continued to mix for another 15 min., big lumps were observed to form in the mixer as shown in the figure.
Hobart mixer of 0.4 ft³ quantity was used to mix the smaller amount of concrete by using the Sikament 686 superplasticizer. 6% by weight of cementitious material of Sikament 686 was used to make the concrete flowable and mixed for a longer time.
4” cube molds made of good quality wood were chosen initially. After demolding the samples they kept in the curing room. Due to constant reuse of wood the cube samples had a thin layer of cement slurry that was peeled off from the sides eventually and presented with poor results.

Figure 128 – Peeling of the concrete layer for 4” cube specimens

Table 53 - Mix design 1 using Sikament 686 Superplasticizer

<table>
<thead>
<tr>
<th>Mix proportion by weight</th>
<th>Cement</th>
<th>Silica fume</th>
<th>Sand</th>
<th>w/cm</th>
<th>S.P</th>
<th>Steel fiber</th>
</tr>
</thead>
<tbody>
<tr>
<td>Kg/m³</td>
<td>900</td>
<td>135</td>
<td>1125</td>
<td>207</td>
<td>54</td>
<td>160</td>
</tr>
<tr>
<td>Lb/ft³</td>
<td>56.16</td>
<td>8.424</td>
<td>70.2</td>
<td>12.92</td>
<td>3.36</td>
<td>9.98</td>
</tr>
</tbody>
</table>
4X8” Cylinder were used with the design mix mentioned above and with available machines to make the surface flat. Due to uneven bottom surface the results were bad.

Figure 129 - Cylinder having uneven bottom surface

Table 54 - 28 days strength results of cylinders for mix 1

<table>
<thead>
<tr>
<th>Sample</th>
<th>Strength (psi)</th>
<th>Load (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>10833</td>
<td>136062</td>
</tr>
<tr>
<td>2</td>
<td>9327</td>
<td>117147</td>
</tr>
<tr>
<td>3</td>
<td>8458</td>
<td>106232</td>
</tr>
</tbody>
</table>
Figure 130 - Failure of the cylinder with uneven surface

Trial Mix 2

By maintaining the same design mix different sizes of the sand were used, two different cements were used and high range water reducing agent, Type III Cement + #100 sand + 2% steel fiber is used 4” cubes were used for testing

Table 55 – Compressive strength of Mix 2

<table>
<thead>
<tr>
<th>No</th>
<th>Strength (psi)</th>
<th>Load (lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>13800</td>
<td>220800</td>
</tr>
<tr>
<td>2</td>
<td>13736</td>
<td>219790</td>
</tr>
<tr>
<td>3</td>
<td>14223</td>
<td>227570</td>
</tr>
</tbody>
</table>

Although the results were good, at higher strength, high load carrying capacity testing machine is required.
In the next case #50 and #100 sand was used and only type 1 cement is used with 4” cubes were used for testing.

Table 56 – Compressive strength of #50, #100 used sand

<table>
<thead>
<tr>
<th>No</th>
<th>Strength</th>
<th>Load</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>12768</td>
<td>204290</td>
</tr>
<tr>
<td>2</td>
<td>11765</td>
<td>188240</td>
</tr>
<tr>
<td>3</td>
<td>11289</td>
<td>180624</td>
</tr>
</tbody>
</table>

Every time there was a variation in results due to peeling of side surfaces of the cubes and due to uneven bottom surface of the cylinders.

Trial Mix 3:

Table 57- Design mix proportion of trial mix 3

<table>
<thead>
<tr>
<th>Material</th>
<th>Lb/ft³</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>47.5</td>
</tr>
<tr>
<td>Silica fume</td>
<td>15.4</td>
</tr>
<tr>
<td>Quartz sand</td>
<td>68.0</td>
</tr>
<tr>
<td>Crushed quartz</td>
<td>14.1</td>
</tr>
<tr>
<td>Water</td>
<td>15.2</td>
</tr>
<tr>
<td>Superplasticizer</td>
<td>0.94</td>
</tr>
<tr>
<td>Steel fiber</td>
<td>10</td>
</tr>
</tbody>
</table>

Compressive strength experiment was conducted with trial mix 3, with fibers and without fibers. Moist curing and heat curing for 2 days at 200°C were used.

Results of the compressive strength at 7 days are reported in Table 57.
Table 58 – Compressive strength of Mix 3

<table>
<thead>
<tr>
<th>Specimen no</th>
<th>No fiber (psi)</th>
<th>2% fiber (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>7 Days</td>
<td>28 Days</td>
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<tr>
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<tr>
<td>2</td>
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<tr>
<td>3</td>
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<td>13937</td>
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</table>

Table 58 shows the compressive strength results of heat cured samples. After 24 hours samples were demolded and cured them at 200°C for 48 hours.

Table 59- Compressive strength at 2 days

<table>
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<tr>
<th>Specimen no</th>
<th>No fiber (psi)</th>
<th>2% fiber (psi)</th>
</tr>
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<td>2</td>
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<td>3</td>
<td>31280</td>
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</table>
**Fiber Reinforced Concrete Beam Calculations**

Area of the rebar = 0.4 in$^2$

Compressive Strength of the concrete ($f'_c$) = 16.8 ksi

Yield strength of the rebar ($f_y$) = 60 ksi

Width of the section (b) = 12 in

Total depth of the section (h) = 3 in

Effective depth of the section (d) = 2.5 in

Depth of the neutral axis

\[ c = \frac{\sigma_t bh + A_s f_s}{0.765 f'_c b + \sigma_t b} \]

Ultimate strength of fiber reinforced concrete \( (\sigma_t) \)

\[ (\sigma_t) = \alpha_0 \times V_F \times \sigma_f \times \alpha_b \]

Where \( \alpha_0 \) = orientation factor and is equal to 0.41

\( \alpha_b \) = bond efficiency factor whose value varies from 1 to 1.2 depending on fiber characteristics. For straight fibers value is taken as 1

\( V_F \) = volume fraction of the fibers (2%)

\( \sigma_f \) = tensile strength of the fibers (1000Mpa (145 ksi))

Ultimate moment capacity

\[ M_u = T_s \times Z_1 + T_f \times Z_2 \]

\( T_s \) = tensile force carried by steel bars

\( T_f \) = tensile force carried by steel fibers

Ultimate strength of fiber reinforced concrete \( (\sigma_t) \)

\[ (\sigma_t) = 0.41 \times 0.02 \times 145 \times 1 = 1.19 \text{ ksi} \]
Depth of the neutral axis (c) = \frac{1.189 \times 12 \times 3 + (0.4 \times 60)}{0.765 \times 16.8 \times 12 + (1.189 \times 12)} = 0.39''

Compressive force corresponding to the area of the rectangular block (C_1) = 0.68*f'c*c*b

\[ C_1 = 0.68 \times f'c \times c \times b = 0.68 \times 16.8 \times 0.396 \times 12 = 54.28 \text{ kip} \]

Compressive force corresponding to area of triangular block (C_2) = 0.085 \times f'c \times c \times b

\[ C_2 = 0.085 \times 16.8 \times 0.396 \times 12 = 6.78 \text{ kip} \]

\[ y_1 = (0.80 \times c)/2 = 0.4 \times c = 0.4 \times 0.396 = 0.158 \]

\[ y_2 = (1/3) (0.20xc) + 0.80c = 0.867c = 0.867 \times 0.396 = 0.343 \]

\[ y'_c = \frac{C_1 y_1 + C_2 y_2}{\Sigma C} = \frac{54.28 \times 0.158 + (6.78 \times 0.343)}{54.28 + 6.78} = 0.178 \]

\[ Z_1 = d - y'_c = 2.5 - 0.178 = 2.32 \]

\[ Z_2 = \left(\frac{h-c}{2}\right) + (c - y'_c) = 1.517 \]

\[ T_s = A_s \times f_y = 0.4 \times 60 = 24 \text{ kip} \]

\[ T_f = \sigma_t \times b \times (h-c) = 1.19 \times 12 \times (3-0.39) = 37.27 \text{ kip} \]

Ultimate moment capacity \( M_u = T_s \times Z_1 + T_f \times Z_2 \)

\[ = 24 \times (2.32) + 37.27 \times (1.517) \]

\[ = 9.35 \text{ kip-ft} \]

**Experimental Investigation**

\[ \text{Moment (M)} = \frac{W \times L}{4} = 9 \text{ kip-ft} \]

3.8% error obtained from analysis by ACI 544 and Experimental Investigation.
Table 60- Load vs Deflection for conventional concrete

<table>
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<th>Deflection</th>
<th>Load</th>
<th>Deflection</th>
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Table 61 - Load vs Deflection for UHSC

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</table>
Impact Resistance Test Calculations

Energy = m.g.h

= (300/2.2)*9.81*(4*0.304)

= 1.62 KJ for a height of 4ft

Energy = m.g.h

= (300/2.2)*9.81*(8*0.304)

3.25 KJ for height of 8 ft