COMPRESSIVE STRENGTH AND ELASTIC PROPERTIES OF SELF-COMPACTING CONCRETE USING MICROWAVE-INCINERATED RICE HUSK ASH AND PULVERIZED FLY ASH

KATLEGO KGATALE RAPULA NGOEPE

CIVIL ENGINEERING
UNIVERSITI TEKNOLOGI PETRONAS
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by

Katlego Kgatale Rapula Ngoepe

Dissertation submitted in partial fulfilment of
the requirements for the
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Universiti Teknologi PETRONAS
Bandar Seri Iskandar
31750 Tronoh
Perak Darul Ridzuan

CERTIFICATION OF APPROVAL

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Approved by,

____
(Name of Main Supervisor)

UNIVERSITI TEKNOLOGI PETRONAS

TRONOH, PERAK

December 2009

CERTIFICATION OF ORIGINALITY

This is to certify that I am responsible for the work submitted in this project, that the original work is my own except as specified in the references and acknowledgements, and that the original work contained herein have not been undertaken or done by unspecified sources or persons.

KATLEGO KGATALE RAPULA NGOEPE

ABSTRACT

Self-compacting concrete (SCC) is on average a new kind of high performance concrete that sees the increase the workability and rate of flow with the addition of a superplasticiser. By its very nature, SCC achieves compaction by means of its own weight into the every part of the mould or formwork without segregation of coarse aggregation. Mechanical properties of hardened SCC are considered to be the same as those of OPC concrete of the same W/B (water-to-binder ratio). SCC usually produces greater high-performance properties such as high strength and durability. Improved working environment, faster construction, less remedial work, and increasing overall productivity are the benefits to the construction industry from the utilization of self-compacting. Rice husk is one of the bulky wastes which are abundantly available everywhere in the world at no cost.

Microwave burning to produce MIRHA will convert it into much value for use in concrete as a partial cement replacement material at very low cost. During to the controlled process of burning, emission of greenhouse gases to the environment is eliminated. The study of this report is to evaluate the tensile and compressive strengths of the Self-compacting concrete with the incorporation of Microwave Incinerated Rice Husk Ash (MIRHA) and Pulverized Fly Ash (PFA). Due to the varying strength of concrete with the incorporation of RHA and PFA, the compressive strength will be observed in 3, 7, 28 and 90 days. The tensile strength and modulus of elasticity are to be evaluated to evaluate the elastic behaviour of SCC.

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CHAPTER 1

INTRODUCTION

1.1 Background of Study

Self-compacting concrete (SCC) is a newly discovered type of concrete product that significantly increase the ease and rate of flow due to the addition of stabilizer and superplasticizer to the mix of concrete. As the name suggests, SCC does not require vibration and compaction during the pouring process due to its nature and composition. SCC achieves compaction by means of its own weight without bleeding or segregation of the aggregate in every part of the mould. It can be used in pre-cast applications or for concrete placed onsite. SCC remains advantageous in terms of durable concrete structures, and saves labour, reduction of consolidation noise; high workability; ease of flow around various kinds of reinforcements; and longer life to moulds and improved finishing characteristics. Without any need for vibrating equipment, workers are saved from the exposure of vibration.

SCC was first introduced and specified in Japan in 1980's in order to obtain sturdy concrete structures. Avoiding vibration is not the principal reason for the development however; the commencing point is a growing concern about difficulties of assuring the quality of complex concrete structures because of indigent compaction of in-situ concrete. This leads to jeopardized long-term stability of structures and increased construction costs. The only feasible choice was to remove the dependence of concrete compaction on concrete workers and to replace it with the ability of concrete itself to guarantee the full filling of formwork, perfect compaction, and the full encapsulation of all reinforcing bars.

SCC is a general term for mix design that differ from that of conventional concretes at the molecular interface between the cement compounds and the admixtures polymers. The fluidity of SCC ensures a high-level of durability and workability whilst the quick rate of placement provides an enhanced surface finish. SCC is definitely the way forward for both in-situ and precast concrete industry.

1.2 Problem Statement

Concrete is greatly utilized as a building material. In modern construction, the strength of concrete is increased to ultra high level such as 100MPa and above which usually poses low water-content and workability. The low workability may affect the quality of hardened concrete, such as the possibility of honey-combing, large porosity, etc. SCC offers to resolve such concerns relevant to the quality of the hardened concrete. This project aims to find the elastic behavior of self-compacting concrete due to incorporation of Rice-husk ash (MIRHA) and Pulverized Fly Ash (PFA) as fillers.

1.3 Objectives

This study was undertaken to achieve the following objectives:

- To determine the tensile strength and modulus of elasticity of Self-compacting concrete made from MIRHA and PFA
- To determine the effects of PFA and MIRHA on the compressive strength of SCC
- To find optimum water-cement ratio required to produce a high compressive strength

1.4 Scope of Study

The scope of this study will cover the elastic properties of self-compacting concrete. The concrete will be added with admixtures, namely rice husk ash (MIRHA) and pulverized fly-ash (PFA). The different admixtures will be added in order to study performance of SCC in different water-cement ratio. Also, the incorporation of superplasticiser will be kept at 3%.

All the preparations of the mixtures and materials preparations will take place in the concrete laboratory, except the preparation of MIRHA. The MIRHA was purchased from Bernas Factory at Sungai Renggam. The grinding process of MIRHA was done in Block J (Old Campus building), using Los Angeles Abrasive Machine. PFA will be obtained from the concrete laboratory.

To carry out the investigations, 11 mixtures will be prepared. These mixtures will be divided into 4 mixtures of PFA only; 3 mixtures of MIRHA; 3 mixtures of PFA and MIRHA; and a control mixture. Each mixture consists of 3 beams, 3 cylinders and 12 cubes of 150mm x 150mm x 150mm. The cylinders will be tested after 90 days to obtain tensile strength of the concrete. All cubes from each mixture are divided evenly for four ages namely: 3, 7, 28 and 90 days in order to get the compressive strength.

CHAPTER 2

LITERATURE REVIEW

2.1 SELF-COMPACTING CONCRETE

Self-compacting concrete (SCC) is specified in a manner that no additional inner or outer vibration is required for the compaction. Compaction of SCC is achieved into every part of the mould by means of its own weight. Compaction plays a consequential function in the gradual growth of hardened concrete properties. When particular properties are measured for performance of concrete structures, it is supposed that concrete is well-compacted and homogenous. The purpose of compaction is to achieve the highest possible density. Vibration, which is still the most common way of compacting concrete, has fluidifying results on the mortar constituents of the mix so that internal friction is decreased and dense packing of concrete aggregate takes place. But compaction via vibration is a discontinuous process resulting in hardened concrete with uneven compaction and therefore with different mechanical and durability properties

SCC composition is similar to the conventionally vibrated concrete (CVC), but high amount of superplasticizer for the reduction of liquid limit and for satisfactory workability has been taken into consideration. Thus, in principle, the properties of fresh and hardened SCC, which is contingent upon the mix-design, should not differ from properties of CVC. One condition is only the consistency where SCC have a slump flow $s_f > 65$cm. compactibility. Currently, the procedure for the production of SCC is mainly empirical.

The SCC mix design should be performed so that the prescribed predefined properties of the fresh and hardened concrete are reached as desired. The components should be coordinated in such a way that segregation, bleeding and sedimentation are prevented.

Hydration of Self-compacting concrete

In precisely chemical terms hydration is a reaction of a dehydrated compound with water, producing a new compound known as a hydrate. In cement chemistry, hydration is known as a reaction of nonhydrated cement or one of its ingredients with water, associated with both chemical and physical changes of the process, in particular with the hardening and setting time. The hydration process of cement and its kinetics can be considered to be affected by a variety of factors such as:

- The phase composition of cement
- The fineness of the cement
- The Curing temperature
- The presence of admixtures and foreign ions
- The water-cement (w/c) ratio

Requirements for fresh SCC mixes

The practical requirements of fresh SCC are differ from those of CVC. SCC is a liquid particle suspension and exhibits very different properties in its plastic state. The following properties define the compliance with self-compactability:

Filling Ability – The ability of fresh concrete to completely fill the formwork; and encapsulate reinforcement inserts and substantial horizontal and vertical flow of the concrete within the formwork while maintaining homogeneity. This ability is achieved when fresh mix flows under its own weight. It is the characteristics often referred to as 'fluidity'. Filling ability is normally measured by either slump flow (Figure 2.1) or J-Ring (Figure 2.2) tests. Slump flow is often the only test to verify the compliance with the specification of the fresh concrete which has been delivered on-site.

The filling ability must be high enough to allow air that was trapped during mixing to escape; leaving a well-compacted concrete. Depending on the application, the slump flow values can vary from 550mm (for precast and flat applications) to 850 mm. The results are critically dependent on the proper usage of the equipment.

Figure 2.1: Slump Flow test

The categories of filling ability depending on the slump-flow (SF) spread are:

- SF1 550 – 650 mm low filling ability: The minimum level, which provides self-compaction of the mix, is found within the range. The precise value of any minimum level depends on individual mix design. Fresh mixes with slump-flow spread SF < 600mm are not self-compacting in practical experience.
- SF2 660 – 750 mm good filling ability: Concrete can be used for most practical uses.
- SF3 760 – 850 mm high filling ability: Mixes with this filing ability usually flow very easily (low or zero yield value) and rapidly. Performance may be needed in casting of very complex structures or heavily reinforced concrete structures. During pre-casting, the time is reduced. Due to such filling ability, segregation resistance and passing ability of concrete requires attention in ensuring that it produces homogeneous concrete and remains self-compacting when placed.

Categories of filling ability depending on J-ring spread SF_J are the same to the categories of SF1-3 as described.

- SF_J 1 550 – 650 mm low filling ability
- SF_J 2 660 – 750 mm good filling ability
- SF_J 3 760 – 850 mm high filling ability

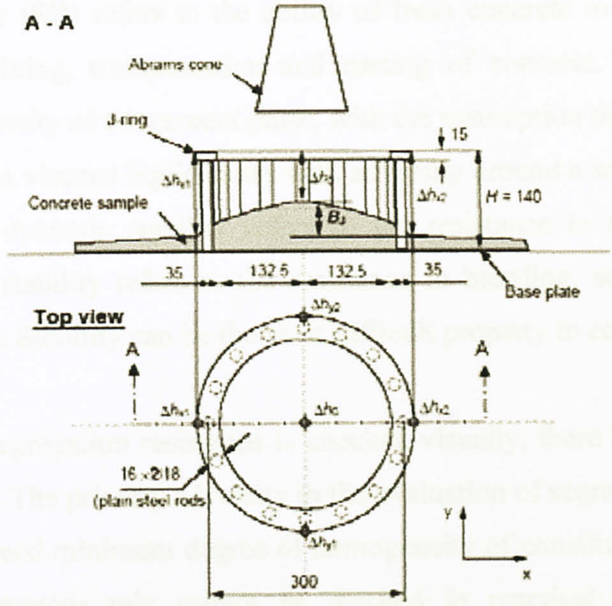

Figure 2.2: J-Ring test

Passing Ability - Passing of obstructions such as narrow sections of the formwork and rigidly spaced reinforcement without blocking caused by interlocking of aggregate particles. Passing ability is usually measured by L-Box (Figure 2.3).

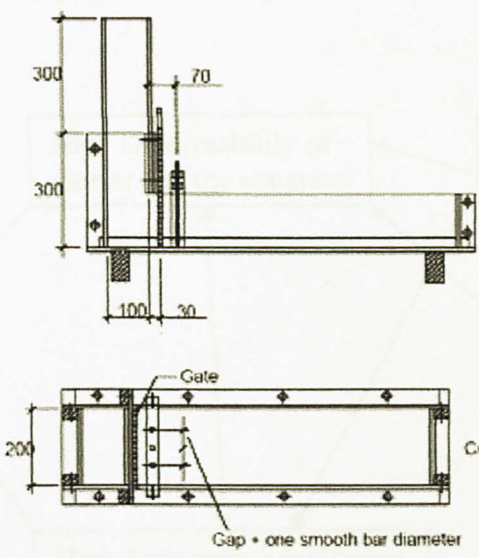

Figure 2.3: L-Box test

Segregation resistance (SR) refers to the ability of fresh concrete to preserve homogeneity (uniformity) during mixing, transportation and casting of concrete. SR also involves the density and plastic viscosity of the cement paste, with the assumption that a solid denser than a liquid tend to sink and a viscous liquid flows with adversity around a solid. SR is also referred to as 'stability'. The dynamic stability refers to the resistance to segregation throughout placement. The static stability refers to the resistance to bleeding, segregation, and surface settlement after casting. Stability can be the most difficult property to compute.

Although the segregation resistance is checked visually, there have been a number of attempts to compute it. The primary adversity in the evaluation of segregation is the absence of a benchmark or an agreed minimum degree of homogeneity of constituents of a concrete mix. A completely homogeneous mix cannot be reached in practical construction. Different construction procedures combined with different demands on the SCC when hardened; clearly require different degree of segregation resistance in addition to appropriate levels of passing ability and filling ability. The segregation resistance differs between:

- Minimum resistance: required for casting simple, lightly reinforced concrete elements.
- Maximum resistance: required for casting complex thin wall elements with congested reinforcement

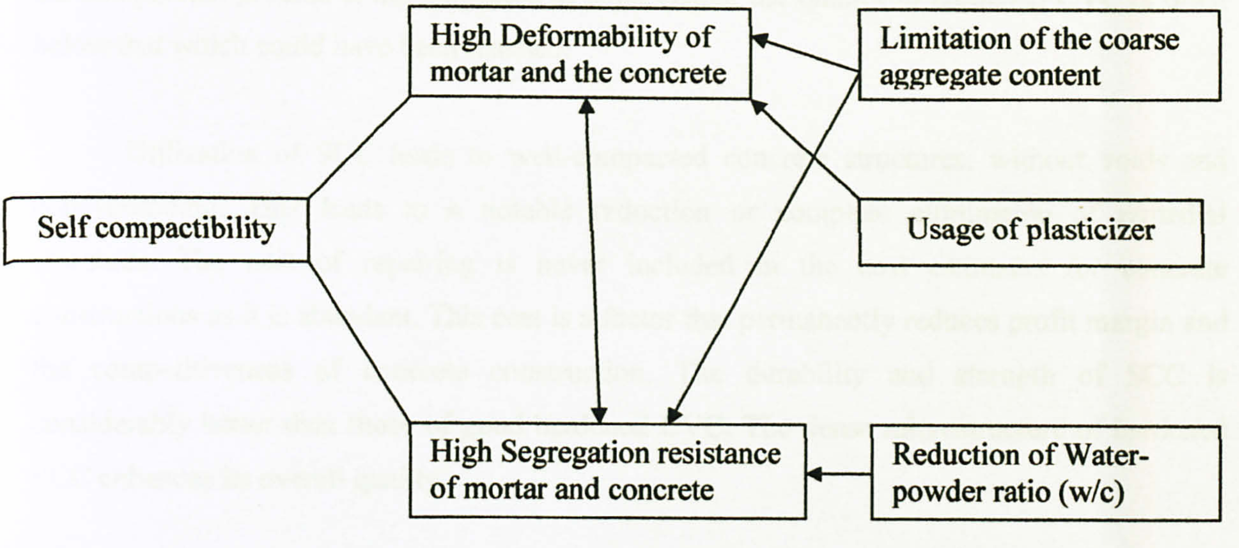

Figure 2.4: *Basic Principles for the production of Self-Compacting Concrete*

SCC is a solid and liquid-particle suspension. Sustaining the flowability of the suspension and to avoid the segregation of the phases is a challenge. The main mechanism to restrain the flowability and firmness of SCC is related to the surface chemistry. Thus, gradual growth of SCC is contingent upon surface-active admixtures as well as how particles are packed.

QUALITY AND ECONOMY

Quality

The casting and compaction of fresh concrete is identified as the most physically demanding and unpleasant activity in the concrete construction project. In some countries, it is difficult to recruit workers that are prepared to do such kind of activity. Compaction process is therefore done by a changeable workforce, often not well trained, but responsible for executing the process.

The quality of completed concrete products depends not only on the properties of the concrete and the process used for production. But it also depends on the training and skills of workers involved, and on their supervision and overall site control. Compaction of conventional concrete is the most undesirable and physically demanding duty in the concrete construction, which means that the least trained construction workforce can carry it out. Since the compaction process is not subjected to supervision, the quality of hardened CVC is often below that which could have been reached.

Utilization of SCC leads to well-compacted concrete structures, without voids and honeycombing. This leads to a notable reduction or complete elimination of remedial measures. The cost of repairing is never included in the cost estimates for concrete constructions as it is abundant. This cost is a factor that permanently reduces profit margin and the competitiveness of concrete construction. The durability and strength of SCC is considerably better than those of good hardened CVC. The dense microstructure of hardened SCC enhances its overall quality.

Direct Costs

The use of SCC affects numerous components of overall construction costs. This leads to both escalation and reduction of direct costs. The reductions are:

- The cost of labour is minimized by removing labourers to compact freshly placed concrete, and their observation. A further reduction in the cost of labour may occur from the reduction of manpower required to cast the mix for some application of SCC. Comparisons of labour requirements when a CVC is placed with the required compaction by vibration, and placing of SCC, assisted by just a single person are shown by Figure 2.5 and Figure 2.6.
- The cost of vibrators of all types and the costs of running them, such as the supply of energy by electricity or compressed air are entirely excluded
- The formwork, namely for precast concrete production can be lighter and more re-uses are possible. The cost of formwork can be reduced.


Figure 2.5: *Placing of CVC by labourers*

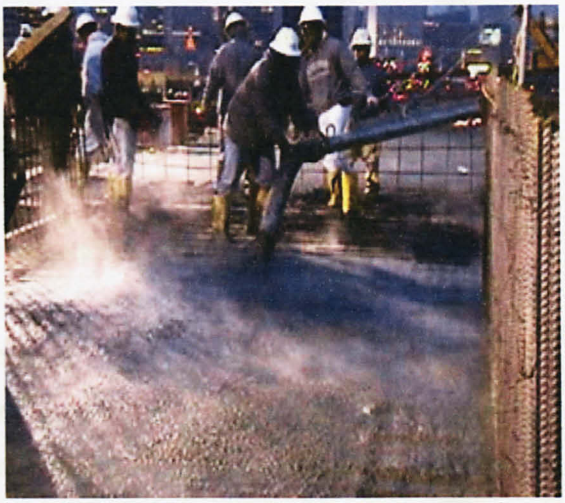
Figure 2.6: Placing of SCC by one worker

- Placing of SCC can be faster than CVC, which requires compaction. Greater and longer nonstop pours become technically viable, while minimal casting time substantially increases productivity of precast yards/ plants of all types. The cost of labour and equipments required for repairs or remedial measures are greatly reduced or completely removed.

- The cost of labour and materials required for remedial purposes are significantly reduced or removed completely.
- Health and safety risks due to exposure of noise and vibrations are reduced

The increases arising out of the adoption of SCC include:

- The cost of constituent materials and of the fresh SCC as supplied tends to be higher than that of a CVC of the same strength grade. This may reflect one or more of factors such as expensive admixtures. The cost can be reduced through long-term contracts or bulk purchases.

Indirect costs

A switch to SCC consists of both positive and negative effect on indirect costs. The reductions are:

- Lower costs of replacement of mould and plant; and lower energy consumption.
- Better working environment on and around construction site. There are lesser direct and indirect health problems; lest time lost through injuries; and minimized chances of injury-related compensations being required.
- Better communications on-site reduce misunderstandings and consequent remedial actions.
- Longer production periods are possible due to bettered working environment

The increases arising out of the adoption of SCC include:

- The costs of additional education and training of existing staff at all levels, from the engineer to the labourer. It is expected that future new staff, at all levels, would have the necessary education and training which include technology of SCC, and the type of indirect cost will be reduced with time.
- New test equipments may have to be purchased by contractors and concrete suppliers.
- Producers may have to invest into extra silos to store various additives and altering of moisture content of aggregate.

Design and management

The addition of SCC as a substitute for CVC provides a number of benefits. However, the optimal benefit is reached when the adaption of SCC is used in the early stages of design and construction process itself. The conditions for a construction project can be specified at different phases of a project, depending on the required benefits of SCC. Such conditions of SCC will differ in detail, which reflects the stage at which it is composed. Similarly to pre-cast concrete, SCC can be produced at a construction site. However, it is usually ordered by the main contractor, giving the supplier the responsibility for the mix design which must give a satisfactory performance in both fresh and hardened state. It is imperative for the supplier or manufacturer of SCC to know detailed information regarding the proposed application of SCC in order to produce an optimum mix. If a proposed application is known, the benefit of using SCC will mainly mitigate the losses replacement of poor quality, remedial works, penalties for late completion by the contractor, etc.

2.2 MICROWAVE-INCINERATED RICE HUSK ASH (MIRHA)

Rice husks are by-products of the paddy milling industries. For rice growing countries, rice husks have attracted more attention due to the environmental pollution and an increasing interest in conservation of energy and resources.

Proper Disposal

About 20% of dried rice paddy is made up of the rice husks. The current world production of paddy is around 500 million tons and hence 100 million tons of rice husks are produced, as shown in Table 2.1. The rice husk has a large dry volume due to its low bulk density (90-150 kg/m^3), and possesses rouge and abrasive surfaces that are highly resistant to natural degradation. Disposal has become a challenging problem. It is recognized that only cement and concrete industries can consume such large quantities of solids pozzolanic wastes.

Table 2.1: World Production Rate for Rice Paddy and Rice Husk (Million Metric Tons)

COUNTRY	RICE PADDY	RICE HUSK
Bangladesh	27	5.4
Brazil	9	1.8
Burma	13	2.6
China	180	36.0
India	110	22.0
Indonesia	45	9.0
Japan	13	2.6
Korea	9	1.8
Philippines	9	1.8
Taiwan	14	2.8
Thailand	20	4.0
US	7	1.4
Vietnam	18	3.6
Others	26	5.2
Total	500	100

Supplementary Binder

For developing countries where rice production is abundant, the use of rice husk ash to partially substitute for cement is attractive because of its high reactivity. As the production rate of rice husk ash is about 20% of the dried rice husk, the amount of MIRHA generated yearly is about 20 million tons worldwide. Also, properly treated ashes have been shown to be active within cement paste. Hence, the use of MIRHA in concrete is important.

Classification of Rice Husk Ash

The chemical composition of rice husk is similar to that of many common organic fibers and contains: a. cellulose ($C_5H_{10}O_5$), a polymer of glucose bonded with B-1.4, b. lignin ($C_7H_{10}O_3$), a polymer of phenol, c. Hemicellulose, a polymer of xylose bonded with B-1.4 whose composition is like xylem ($C_5H_8O_4$), and d. SiO_2, the primary component of ash. The holocellulose (cellulose combined with hemicelluloses) content of rice husk is about 54%, but the composition of ash and lignin differ slightly depending on species. After burning, most evaporable components are slowly lost and silicates are left. The characteristics of ash are dependent on the components, temperature and time of burning.

In order to obtain an ash with high pozzolanic activity, the silica should be held in a non-crystalline state and in a highly microporous structure. Hence, the burning process should be controlled too remove the cellulose and lignin portion while preserving the original cellular structure of rice husk. Traditional open-field burning can create air pollution that is suspected to cause lung and eye diseases within human population, as well as damage to plant life. [1]

The effect of Burning Temperature

(a). At 400°C, due to transglycosylation, polysaccharides begin to depolymerize, producing levoglucan, monosaccharide derivatives, and oligosaccharides.

(b). Dehydration of the sugar units occurs above 400°C producing 3-deoxyglucosenone, levoglucosenone, funicular and furan derivatives.

(c). At 700°C, the sugar units decomposes, producing some cabby compounds such as acetaldehyde, glyoxal and acrolein.

(d). At temperature above 700°C, these unsaturated products react together and through free radical reaction, form a highly reactive carbon residue.

(e). Because of the clinkering temperature of rice husk is under 1500°C, polymorphism of silica such as quartz, tridymite, and cristobalite with open structured SiO_2 in MIRHA is produced which promotes pozzolanic reaction. However, this reaction may vary with the solubility of, SiO_2, depending on the density of the silica.

Analysis of the quality of MIRHA

The quality of MIRHA actually depends on the method of ash incineration and the degree of grinding. It also depends upon the preservation of cellular structure and the extent of amorphous material within the structure. The diffusion process for obtaining a reactive cellular rice husk is shown in Figure 2.7.

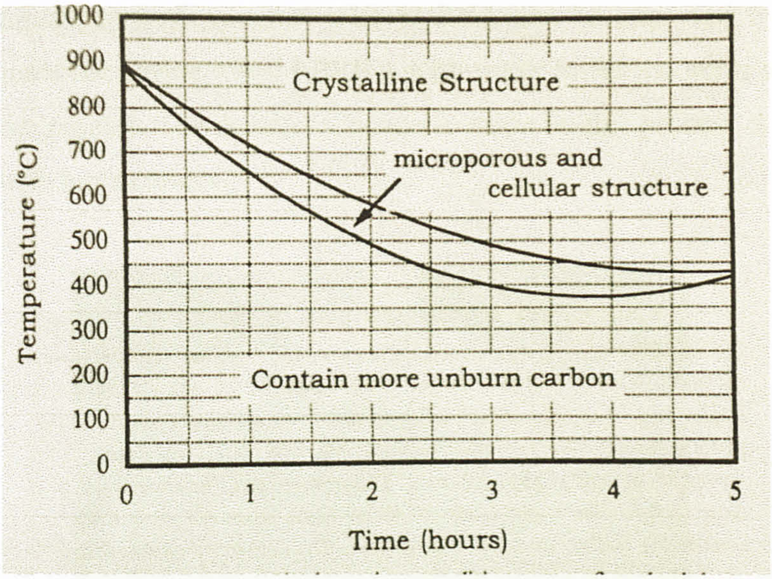

Figure 2.7: *The optimum incineration condition curve for obtaining reactive cellular MIRHA*

EARLY CHARACTERISTICS OF CONCRETE WITH MIRHA

The early strength characteristics of concrete with MIRHA depends on the water to cement content ration, the amount of paste used, the amount of MIRHA added, other admixtures used and mixture proportion.

The Workability of Fresh Concrete with MIRHA

At a given water to cement content ratio, small addition (less than 2 to 3 by weight of cement) of MIRHA may be helpful for improving the stability and workability of concrete by reducing the bleeding and segregation. This is mainly due to the large surface area of MIRHA which is in the range of 50- 60m^2/g. Large additions would produce dry or unworkable mixtures unless water-reducing admixtures or superplasticizers are used. Due to the adsorption character of Cellular MIRHA particles, concrete containing MIRHA require more water for a given consistency. For a given consistency, the reduction of water requirement can lead an overall improvement in many engineering properties.

The workability of fresh concrete with MIRHA can be improved by densifying the mixture. The process uses cement and MIRHA with water to fill the pores and voids within well-compacted aggregates. The density of concrete made by this process is higher than that of conventional with more cement.

Figure 2.8: Microwave Incinerated Rice Husk Ash

Hydration Mechanisms of Paste with MIRHA

The penetration resistance coincides with the growth of Calcium hydroxide (CH) up to 8 hours, and this is similar to the behavior of ordinary Portland cement paste. The early resistance may be primarily due to the formation of CH crystal. The formation of CH at the surface of MIRHA is due to the adsorption by cellular structure of MIRHA. In such cases bleeding water will be significantly reduced. The adsorbed water enhances the pozzolanic reaction inside the cellular spaces and gain significant strength. After 40 hours, the pozzolanic reaction further binds Si in MIRHA with CH to form C-S-H gel and solid structure. This means that MIRHA fills the finer pores and reduces the permeability, which is beneficial to the durability of concrete.

2.3 PULVERIZED FLY-ASH (PFA)

Pulverized fuel ash or known as fuel ash is a commonly used artificial pozzolan derived from the combustion of pulverized coal in furnace of thermal or electric power plant. The characteristics of fly ash vary according to the combustion operation system as well as the coal composition. The influence on the properties of fresh concrete is linked to the shape of the fly ash particles. Most of these particles are spherical and solid. It is often seen under microscopic examination that some particles have enclosed pockets of air, (1) or appear to be hollow spheres spheres, known as cenospheres, or vesicular in shape.

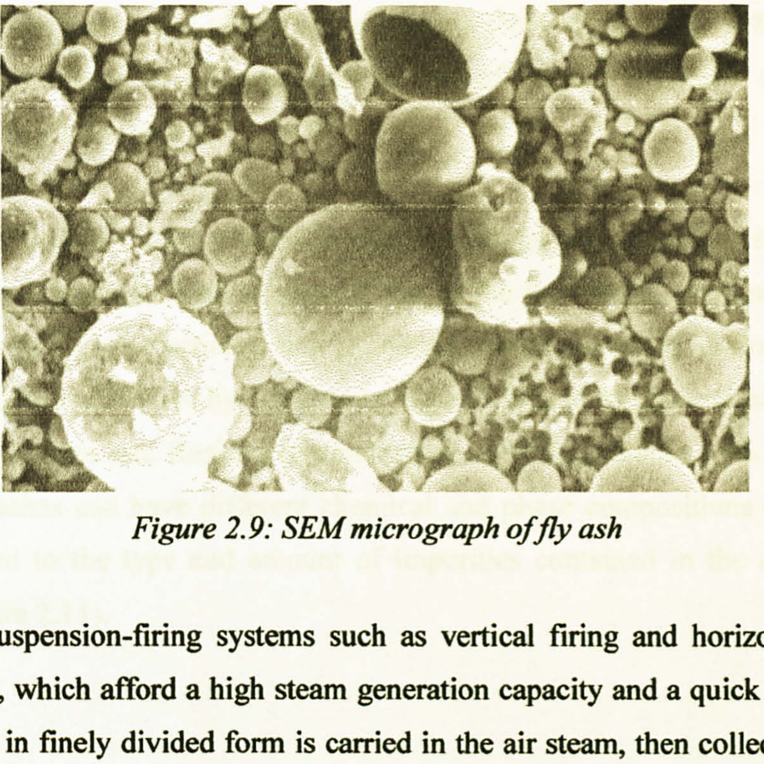

Figure 2.9: SEM micrograph of fly ash

Various suspension-firing systems such as vertical firing and horizontal firing have been widely used, which afford a high steam generation capacity and a quick response to load changes. The ash in finely divided form is carried in the air steam, then collected by electrical or mechanical precipitators (dry process) while it is quickly cooled. Fly ash particles those are collected in electrostatic precipitators are usually silt size (0.074-0.005 mm). In some power stations, the old wet collection processes are still in use (see Figure 2.10)

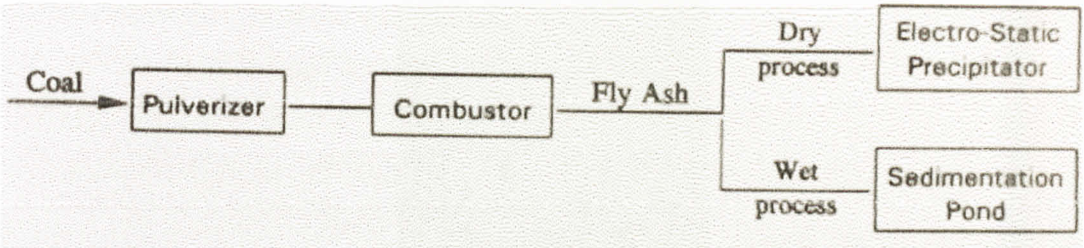

Figure2. 10: Schematic diagram of coal burning and ash collection processes

The amount of crystalline material versus glassy phase material depends largely on the combustion and glassification process used at a particular power plant. When the maximum temperature of the combustion process is above approximately 1200^0 C and the cooling time is short, the ash produced is mostly glassy phase material (McCarthy et. al., 1987). Where boiler design or operation allows a more gradual cooling of the ash particles, crystalline phase calcium compounds are formed.

Fly ash collected by a dry process is usually homogenous in particle size; whereas that collected by a wet process is more segregated due to the fact that the sedimentation speed is lower for smaller or lighter particles, and it contains large quantities of water. The coarser portion of the coal ash (15-20 % by mass) is heavy and falls to the bottom of the furnace and thus is called the bottom ash. Residue of combustion consists of about 85% fly ash and 15% bottom ash. Fly ashes can have different chemical and phase compositions because they are exclusively related to the type and amount of impurities contained in the coal burnt in the power plant (Figure 2.11).

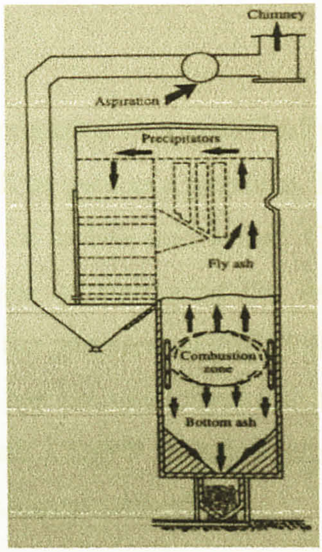

Figure2.11: Schematic representation of the formation of fly ash

Fly ash is a powder with particle sizes similar to that of cement particles. Generally, more than 70% of fly ash able to pass through a 45μm sieve, and a fraction of particles are smaller than 3μm. Being a finely divided powder consisting of silica-alumina glass of various forms, fly ash in concrete works as a filler between cement grains and aggregate, and as a effective binder providing a cementitious properties. In fact, when fly ash is a major constituent of concrete, either by replacing a portion of cement or by replacing a part of fine aggregate, it physically alters the properties of the system. Most importantly, it alters the water to cement ratio (w/c). As stated by Aimin Xu, the water to binder ratio is related to w/c by

This means that when water to binder ratio is the same, the concrete with more fly ash has a higher water to cement ratio. Thus, a 50% replacement leads to a doubled w/c ratio. Since fly ash particles are larger than the cement particles, less water is required to wet the fly ash. However, water is required to wet the same mass of finer ash. The variation of water requirement occurring within each particle size range can be attributed to different morphology and surface area of the ash particles. The water requirement has been shown to be lower with the increment of the fly ash content [(5)]. The reduction in the water demand of concrete caused by the presence of fly ash is due to the spherical shape, which is known as the "ball-bearing effect". A concrete mix containing fly ash is cohesive and has a reduced bleeding capacity. The mix is suitable for pumping.

Classification of fly ash

Although fly ash is typically produced in a coal-fired power plant, it is important to choose the relevant type of ash that will be used as a pozzolan. For example, coal from the east coasts tends to contain high volume of carbon and sulfur. Colour is one of the important physical properties of fly ash in terms of estimating the lime content qualitatively. It is suggested that lighter color indicate the presence of high calcium oxide and darker colors suggest high organic content. Fly ash has been classified into two classes, F and C, based on the chemical composition of the fly ash. Class F fly ash is produced from burning anthracite and bituminous coals, and has higher ultimate strength.

Class F fly-ash has siliceous and aluminous material, which itself possesses little or no cementitious value but will, in finely divided form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperature to form cementitious compounds. If the fly ash has high calcium content, it should not be used in sulfate exposure or hydraulic applications. According to ASTM C 618-94a Standard Specification for Coal Fly Ash, the chemical requirements to classify any fly ash are shown in Table 2.2 and Table 2.3 below.

Table 2.2: Chemical Composition of Class F Fly Ash

Substance or Properties	Requirements (ASTM C 618) %
Silicon dioxide (SiO_2) plus aluminum oxide (Al_2O_3) plus iron oxide (Fe_2O_3), min	70.0
Sulfur trioxide (SO_3), max,	5.0
Moisture Content, max,	3.0
Loss on ignition*, max,	6.0

Class C fly ash is produced normally from lignite and sub-bituminous coals and usually contains significant amount of Calcium Hydroxide (CaO) or lime (Cockrell et. al., 1970). This class of fly ash, in addition to having pozzolanic properties, also has some cementitious properties (ASTM C 618-99). Concrete made with Class C fly ash (as opposed to Class F) has higher early strength due to the presence of lime. This allows pozzolanic activity to begin earlier. At later ages, Class C behaves similar to Class F, producing higher strengths than conventional concrete at 56 and 90 days.

In combination with Portland cement, **Class C** fly ash can be used as a cement replacement, ranging from 20-35% of the cementitious material. Class C must replace at least 25% of the Portland cement to mitigate the effects of alkali silica reaction.

Table 2.3: Chemical Composition of Class C Fly Ash

Substance or Properties	Requirements (ASTM C 618) %
Silicon dioxide (SiO_2) plus aluminum oxide (Al_2O_3) plus iron oxide (Fe_2O_3), min	50.0
Sulfur trioxide (SO_3), max,	5.0
Moisture Content, max,	3.0
Loss on ignition*, max,	6.0

"Loss of ignition" refers to the carbon content available in the ash. The more carbon that is present, the more weight will be lost upon burning process of ash. Ideally, there should not be weight loss. Extent of packing depends on the type of cement and fly ash used. Better packing would be achieved with coarser Portland cement and finer fly ash. One beneficial effect of packing on strength is the reduction in volume of capillary pores. The strength of concrete cured up to 28 days is sometimes lower due to un-reacted fly ash. In such concrete, the un-reacted fly ash weakens the paste-aggregate interfacial zone of a structure (2).

EARLY CHARACTERISTICS OF CONCRETE WITH PFA

PFA has an important effect on the hydration rate as well as the efficiency of the chemical admixtures, particularly air entraining agent and superplasticizer. Low calcium fly ash acts a fine aggregate of circular shape in early stages of hydration whereas high calcium fly ash contributes to the early cementing reactions in addition to its presence as fine particulate in the concrete mix.

Water requirement and workability

In concrete mix design, workability is a growing and governing aspect. Water content of concrete plays an imperative role in governing workability. The spherical shape and glassy surface of fly ash particles allow greater workability or slump for equal water-cement ratios.

Due to the fine particle size and the spherical shape as well as glassy surface, PFA plasticize SCC at given water content when used as a cement or fine aggregate replacement.

Time of Setting

Due to the addition of water in SCC, hydration reaction commences and the cement paste starts to stiffen with the release of heat. The stiffening rate of cement in defined in terms of setting time. Investigations have shown that the addition of low Class F fly ash usually show some extent of retarding effect on the cement setting. The effect of fly ash on setting time depends on the characteristics and quantity of fly ash used. Class C fly ash usually low in carbon and high in cementitious components, tends to reveal opposite behaviour of decreased setting time.

Bleeding and Segregation

Application of fly ash in SCC extensively reduces segregation and bleeding. The lubricating effect of the glassy and spherical particles and the increased ratio of solids to liquid make the concrete less susceptible to segregation and escalate SCC pumpability. Figure 2.12 shows the bleeding rate of fly ash concrete compared to that of control concrete without fly ash.

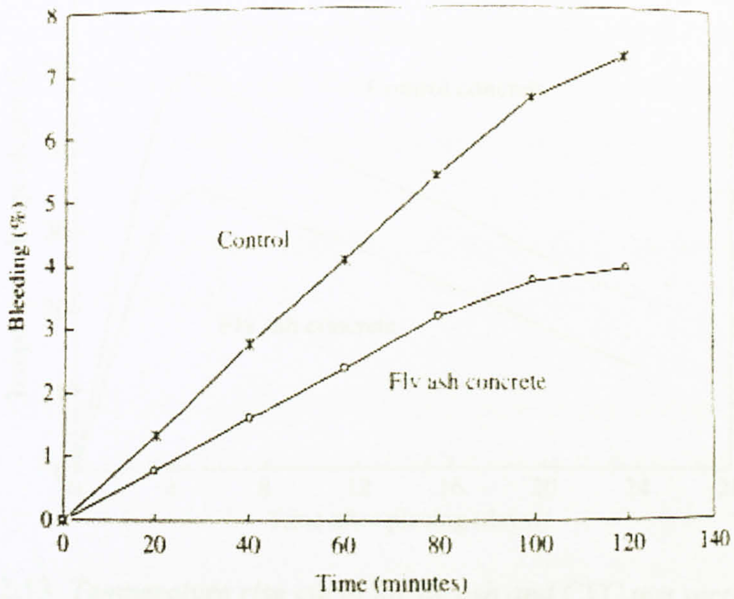

Figure 2.12: *Relative bleeding of control and fly ash concretes (Central Electricity Generating Board)*

Temperature Rise

During hydration process, heat is released which cause a rise in temperature. Since pozzolanic reaction reactions takes are slower, fractional substitution of cement by fly ash results in release of heat over a longer period of time and the concrete temperature remains lower because heat is dispersed as it is evolved. This occurrence is effective in mass concreting, where cooling following a substantial temperature rise due to hydration heat occurs, stresses can develop and cause cracking. The rise in temperature depends on the heat generated during hydration process and pozzolanic reaction, as well as the thermal properties of concrete. Sivasundram (1999) reported that low calcium Class F fly ashes tend to minimize the temperature rise more as compared to high-calcium Class C fly ash.

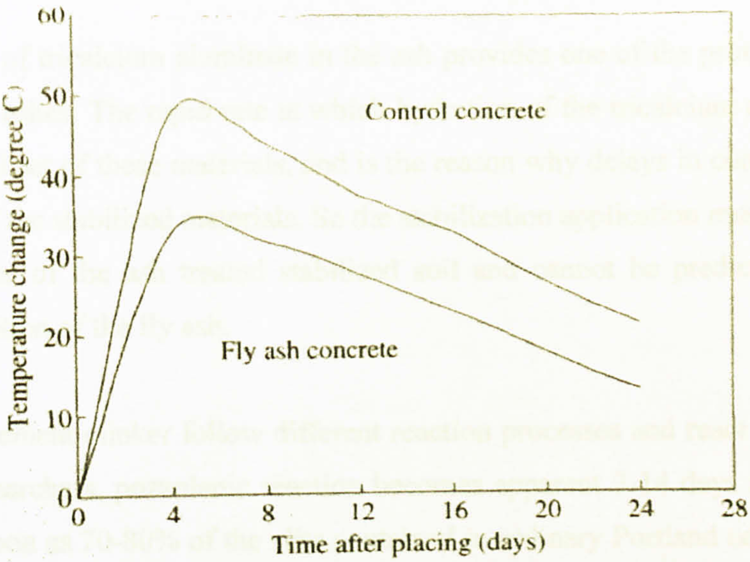

Fig 2.13: *Temperature rise curve for fly ash and CVC test sections*

Hydration Mechanisms of Paste with PFA

Addition of fly ash in concrete, either by substituting a portion of cement or by substituting a portion of fine aggregate, changes the concrete proportion. More important, the water to cement ratio (w/c) becomes altered. Formation of cementitious material by the reaction of free lime (CaO) with the pozzolans (AlO_3, SiO_2, Fe_2O_3) in the presence of water is known as hydration. The hydrated calcium silicate gel or calcium aluminate gel (cementitious material) can bind inert material together.

For class C fly ash, the calcium oxide (lime) of the fly ash can react with the siliceous and aluminous materials (pozzolans) of the fly ash itself. Since the lime content of class F fly ash is relatively low, addition of lime is necessary for hydration reaction with the pozzolans of the fly ash. An important difference between cement and fly ash is the lower amount of CaO content in fly ash, which means that the fly ash has pozzolanic properties instead of hydraulic properties. For lime stabilization of soils, pozzolanic reactions depend on the siliceous and aluminous materials provided by the soil.

Hydration of tricalcium aluminate in the ash provides one of the primary cementitious products in many ashes. The rapid rate at which hydration of the tricalcium aluminate occurs, results in the rapid set of these materials, and is the reason why delays in compaction result in lower strengths of the stabilized materials. So the stabilization application must be based on the physical properties of the ash treated stabilized soil and cannot be predicted based on the chemical composition of the fly ash.

The fly ash and cement clinker follow different reaction processes and react at different rates. According to researchers, pozzolanic reaction becomes apparent 3-14 days after mixing with water. Thus, as soon as 70-80% of the alite contained in ordinary Portland cement has reacted. The rate of the pozzolanic reaction is dependent on the properties of the fly ash and of the mix. The reaction is also depends on temperature and pH-value. The effect of fly ash on the cement binder will not only be the pozzolanic activity, but also the acceleration of the cement. This means the two processes will interfere with each other. The presence of fly ash affects many aspects of hydration:

- The formation of portlandite
- The kinetics of reaction
- The composition of the hydrates

2.4 COMPRESSIVE STRENGTH

Strength of concrete is usually taken into account as the most valuable property. Although in many practical cases, other characteristics, such as durability and permeability could be more important. Strength usually gives an overall picture of the quality of concrete since it is directly related to the structure of a hydrated cement paste or concrete. In general, there are various factors that influence the strength of concrete on a smaller or larger scale. Such factors include:

- The concrete-making procedure, such as mixing, batching, mixing, delivering, placing and compaction of fresh concrete.
- Age during testing

- Wetness of curing, porosity
- Soundness of aggregate
- Water/ cement ratio
- Testing procedures, including the shape and size of the specimen, type of testing machine, etc.

It is anticipated that the strength of concrete increases with the hydration process, that is, with age. When hydration process is completed or stopped, the strength development stops. The presence of water has considerable influence on the behavior of hardened concrete such as: (1) keeping the concrete wet by curing at early ages prevents, (2) the presence of water assists the hydration process, (3) freezing of water within the concrete can seriously hurt it, and (4) free water can affect some other engineering properties of concrete. The duration and magnitude of strength development in a wet-cured concrete is larger than that of dry-cured concrete. However, the compressive strength of a Portland cement concrete is higher when the sample is dry, or the surface layer is dry than a fully or in a fully or surface-saturated sample. Also, the strength of concrete also depends on the cohesion of the cement paste; on its adhesion to the aggregate particles, and to a certain extent on the strength of aggregate itself.

The compressive strength of concrete with MIRHA

In conventional concrete, the transition zone is generally less dense than the bulk paste and contains a large amount of paste-like crystals of calcium hydroxide. This is suspected to induce micro-cracks due to the tensile stresses induced by thermal and humidity change. The structure of the transition zone is the weakest phase in the concrete and has a strong influence on the properties of concrete. The addition of MIRHA can influence both the permeability and strength by strengthening the interface of aggregate and cement paste; and by closing the large voids in the hydrated cement paste through the pozzolanic reaction. This phenomenon is shown in Figure 2.14.

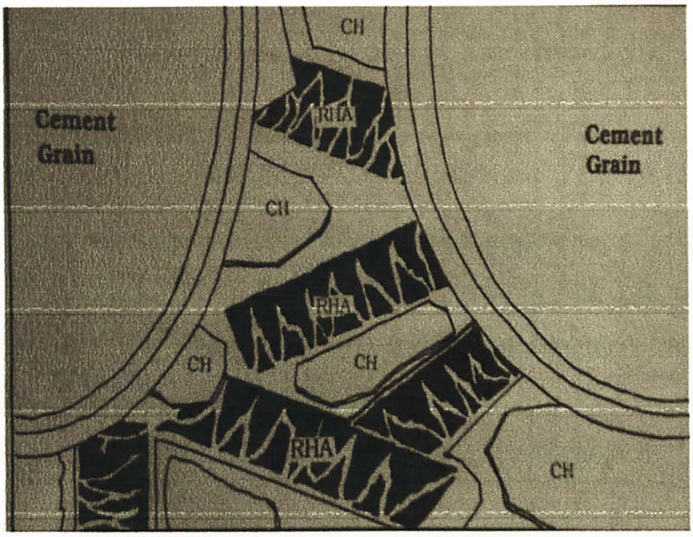

Figure 2.14: Void filling and transition zone strengthening effects of MIRHA

Pozzolanic reaction is known to modify the structure of pores. During the pozzolanic reactions, the empty spaces in the pore-structure of concrete are occupied, thus increasing density. Porosity of the concrete is reduced, and subsequently, the pores are refined. These reactions are slow and proceeds with time. Refinement of pores remains in progress even after 28 days (4)

The compressive strength of concrete with fly-ash

The rate of strength development of concrete incorporated with fly ash depends on various factors such as chemical and mineralogical arrangement, type of cement used, mix proportions, pozzolanic reactivity and curing environment. The highly pozzolanic fly ash contributes to the strength development almost from the beginning of the hydration process of cement. Class F fly ash does not display substantial pozzolanic reaction to influence the strength until about two weeks after hydration process. Class C fly ash with calcium content exceeding 15%, may contribute to the strength development as early as 3 days after hydration due to its self hardening and pozzolanic properties. Due to its fineness and pozzolanic reactivity, PFA in SCC improves the quality of cement paste and the micro-structure of the transition zone between the binder and the aggregate. As a result of a continual process of pore refinement, due to the incorporation of PFA, an increase in strength development with curing age is reached.

2. 5 TENSILE STRENGTH

Direct Tensile strength test

The direct means of calculating the tensile strength of concrete or mortar is the similar to that of metals and other materials. This is true, on the provision that the specimen which is a briquette, long cylinder or prism is subjected to a uniformly allocated increasing tension load in an appropriate testing machine until the specimen breaks into two portions. The strength is specified as the tension load per cross-sectional area, usually measured in MPa.

Gripping of the sample is usually achieved either by trimmed cones or by steel reinforcement pushed into concrete samples. The adversity associated with this way of direct tension test is that it is afflicted with misalignment and clamping stresses. Thus the stress concentrations, bending or torsion reduced the magnitude of the computed strength. Thereby, making the reliability of these strength results arguable. This problem is overcome by reducing the central portion of the strength specimen.

Because of the adversity associated with the direct tensile strength, other procedures have been selected for indirect estimation of the tensile strength. One should expect from these procedures to provide the "true" tensile strength. Tensile strengths obtained indirectly are suitable for comparison of different mixtures, and they are convenient because of the ease of the tests.

Splitting Tensile test

The known procedure of computing the tensile strength was developed in Brazil in Carneiro and Barcellos (1949, 1953). Initially a cylindrical sample was detailed and standardized in ASTM C496, but the procedure has been successfully been applied on cubes, as Figure 2.15 shows. Among these procedures the diagonal split cube method is the least suitable, but can provide a stress-strain diagram. In the splitting test a vertical compressive load is applied uniformly as a line on two opposing generatrices of the sample, as shown in figure 2.15.

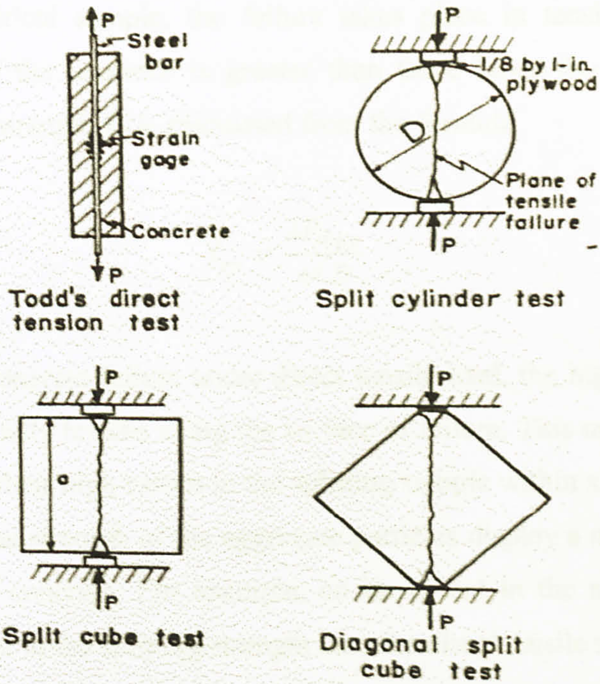

Figure 2.15: *Direct and Indirect methods for determination of tensile strength of concrete*

The compressive load adapted on the generatrices creates a compressive and tensile stresses in the sample. The maximum stresses are situated on the elements located in the internal vertical surface between the loaded generatrices. The horizontal stresses are compressive near the loaded generatrices but become tensile soon as they move away from the external surface so that an almost uniform tensile stress field exists over 80% of the vertical plane. (Figure 2.16)

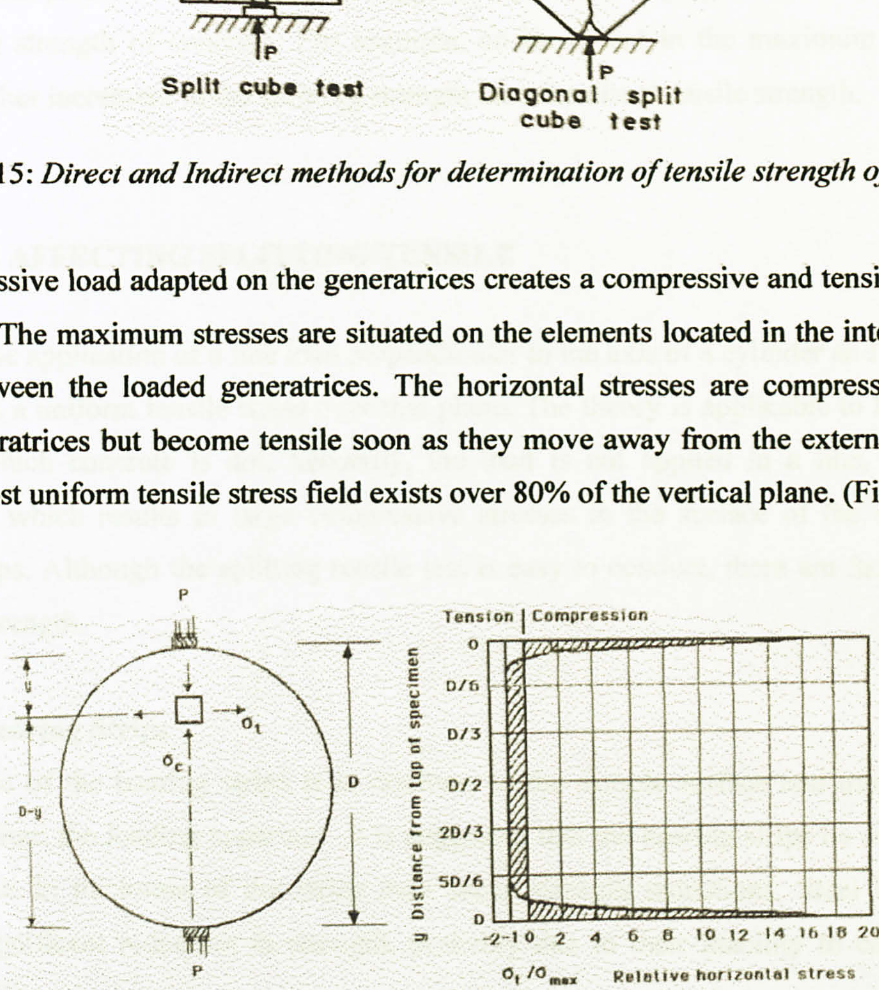

Figure 2.16: *Distribution of Horizontal stresses in the internal surface between the loaded generatrices of a cylinder under splitting test*

In the case of a cylindrical sample, the failure takes place in tension provided that the compressive material of the concrete is greater than three times the tensile strength. The cylinder splitting tensile strength f_{sp} is calculated from the formula

$$f_{sp} = \frac{2P_{max}}{\pi L D}$$

In contradiction to the concrete failure under direct tensile load, the bigger part of the coarse aggregate particles is usually broken along the surface of failure. This may due to the fact that the tensile stresses have their high values in the splitting sample within a narrow strip along the central vertical plane. The strength of the aggregate particles display a noticeable influence on the splitting strength of concrete. For example, an increment in the maximum particle size causes a higher increment in the splitting strength than the direct tensile strength.

FACTORS AFFECTING SPLITTING TENSILE

In theory, the application of a line load perpendicular to the axis of a cylinder and a diametrical plane yields a uniform tensile stress over that plane. The theory is applicable to homogeneous material, which concrete is not. Secondly, the load is not applied in a line, but is rather distributed, which results in large compressive stresses in the surface of the sample under loading strips. Although the splitting tensile test is easy to conduct, there are factors found to affect the strength.

Effect of Bearing Strips

The purpose of the bearing strips is to conform to the sample surface and transfer the load uniformly from the loading apparatus. It is suggested that the bearing strips be made of wood. The increase in thickness of the strips may cause strength reductions. Steel bearing strips shows a significant reduction in strength, probably due to their inability to conform to the sample surface.

Effect of Loading Rate

Just with compressive strength and flexural strength tests, higher splitting tensile strengths are obtained when samples are loaded at a more rapid rate.

Effect of Sample Size and Dimensions

The length of a cylinder for a given diameter does not affect the results, other than producing more uniformity of results for longer samples. However, the samples with smaller diameters seem to yield tensile strengths that are higher than samples with bigger diameters.

CHAPTER 3

METHODOLOGY / PROJECT WORK

3.1 Project Work

The general sequence of my research is as follows:

3.1.1 Literature Review/ Data Collection

In this stage of methodology, the information will be gathered from various books, papers, internet and journals that have already been published. The data is useful for better understanding of the project.

3.1.2 Laboratory Work

After all the relevant data and information is collected, laboratory tests will be conducted to test the performance of concrete with incorporation of MIRHA and PFA. The tests to be done are:

a. Compressive strength test
b. Split-cylinder test
c. Flexural prism test

The above mentioned tests will be done at Concrete Lab in Block 13, Civil Engineering Department. Preparation of MIRHA will be done at Block J.

3.1.3 Data Analysis

All the results from the laboratory tests will be analyzed. Appropriate tables and graphs will be drawn to get a clearer view of the results

3.1.4 Report Writing

The final part of the project is writing the report whereby all the findings and results of this research will be presented.

3.2 Methodology

3.2.1 Tools and equipments

a. Microwave incinerator

b. Los Angeles Abrasion Testing Machine.

c. Concrete mixing machine.

d. Compressive strength testing machine.

e. Universal testing machine

Microwave Incinerator

The preparation of Microwave Incinerated Rice Husk Ash (MIRHA) was prepared at the Incinerator room (Block J). MIRHA is a product from burning process of rice husk. Approximately 10 kg of ash was produced, after burning 100kg of rice husk. For this project, a total amount of 19.1 kg of MIRHA was added to the SCC.

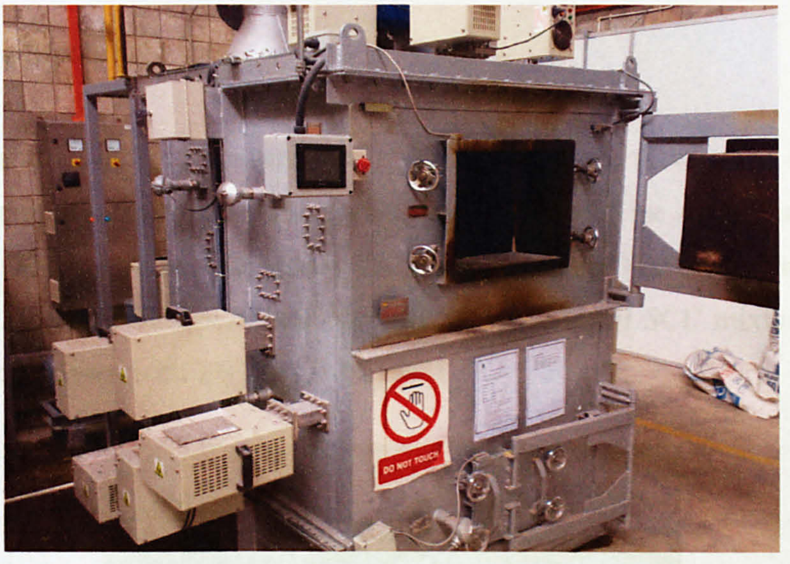

Figure 3.1: *Microwave Incinerator*

Los Angeles Abrasion Testing Machine

Approximately, 6 to 7 kg of grounded MIRHA can be prepared after at least 500 cycles of the grinding process. The testing machine is located at Block J.

Figure 3.2: *Los Angeles Abrasion Testing Machine*

Concrete Mixing Machine

For preparation of both conventionally vibrated concrete (CVC) and self-compacting concrete (SCC) samples, the following ingredients will be used: Fine Aggregate (sand), course aggregate- CA (20-8), course aggregate- CA (8-4), ordinary Portland cement and water. Approximately, 3 percent of superplasticizer will be added to all SCC mixtures. The mixing process was in accordance with BS 1881-108(1983).

Figure 3.3: *Mixing process of concrete*

Compressive strength testing machine

Compressive strength testing machine was used for determining the compressive strength of the concrete specimens according to BS 1881:116: 1983. The equipment was also used for the determination of Split-tensile strength according to BS 1881:117:1983, which specifies the use of hardboard strips. Due to a continuous pozzolanic reaction, the ultimate strength of SCC incorporated with MIRHA and PFA is not limited to 28 days. Specimens were tested at the following ages: 3, 7, 28 and 90 days.

Figure 3.4: *Compressive strength testing machine*

3.2.2 Mix Proportions

Table 3.1: Mix Proportion of Concrete **(Materials in 1m^3)**

Mix No	Poz	OPC	MIRHA	PFA	CA 20-8	CA 8-4	FA	w/b	w/c	water	S/P %	total	Poz. %	coarse agg (%)	fine agg (%)
1	PFA	450	0	50	315	610	850	0.25	0.28	125	3	2400	10	52.11	47.89
2A		450	0	50	310	605	845	0.28	0.31	140	3	2400	10	51.99	48.01
2B		450	0	50	312	605	833	0.3	0.33	150	3	2400	10	52.40	47.60
3		450	0	50	303	600	837	0.32	0.36	160	3	2400	10	51.90	48.10
4	MIRHA + PFA	400	50	50	312	605	833	0.3	0.38	150	3	2400	20	52.40	47.60
5		400	50	50	305	600	825	0.34	0.43	170	3	2400	20	52.31	47.69
6		400	50	50	300	590	820	0.38	0.48	190	3	2400	20	52.05	47.95
7	MIRHA	450	50	0	310	600	815	0.35	0.39	175	3	2400	10	52.75	47.25
8		450	50	0	305	595	810	0.38	0.42	190	3	2400	10	52.63	47.37
9		450	50	0	300	590	800	0.42	0.47	210	3	2400	10	52.66	47.34
10	Control	500	0	0	265	575	810	0.5	0.50	250	0	2400	0		

Table 3.2: Mix Proportion of Concrete **(Material in kg)**

Vol. 0.05879 m^3

Mix No	Poz	OPC	MIRHA	PFA	CA 20-8	CA 8-4	FA	w/b	w/c	water	SP	total	Poz. (%)
1	PFA	26.45	0.00	2.94	18.52	35.86	49.97	0.25	0.28	8.75	0.88	142.49	10
2A		26.45	0.00	2.94	18.22	35.57	49.67	0.28	0.31	8.23	0.88	141.09	10
2B		26.45	0.00	2.94	18.34	35.57	48.97	0.30	0.33	8.82	0.88	141.09	10
3		26.45	0.00	2.94	17.81	35.27	49.20	0.32	0.36	9.41	0.88	141.09	10
4	MIRHA + PFA	23.51	2.94	2.94	18.34	35.57	48.97	0.30	0.38	8.82	0.88	141.09	20
5		23.51	2.94	2.94	17.93	35.27	48.50	0.34	0.43	9.99	0.88	141.09	20
6		23.51	2.94	2.94	17.64	34.68	48.21	0.38	0.48	11.17	0.88	141.09	20
7	MIRHA	26.45	2.94	0.00	18.22	35.27	47.91	0.35	0.39	10.29	0.88	141.09	10
8		26.45	2.94	0.00	17.93	34.98	47.62	0.38	0.42	11.17	0.88	141.09	10
9		26.45	2.94	0.00	17.64	34.68	47.03	0.42	0.47	12.35	0.88	141.09	10
10	control	29.39	0.00	0.00	15.58	33.80	47.62	0.50	0.50	14.70	0.00	141.09	0
total material		285.1	17.64	20.58									

CHAPTER
4

RESULTS AND DISCUSSION

4.1 COMPRESSIVE STRENGTH RESULTS

Table 4.1.1: Average Compressive strength of PFA versus curing age

Average Compressive Strength, MPa				
Day	**3**	**7**	**28**	**90**
PFA 1	38.01	42.43	65.78	63.83
PFA 2A	30.34	49.93	56.14	57.13
PFA 2B	53.53	57.65	65.63	70.54
PFA 3	37.06	39.14	61.02	64.76
Control	38.53	47.45	49.02	53.70

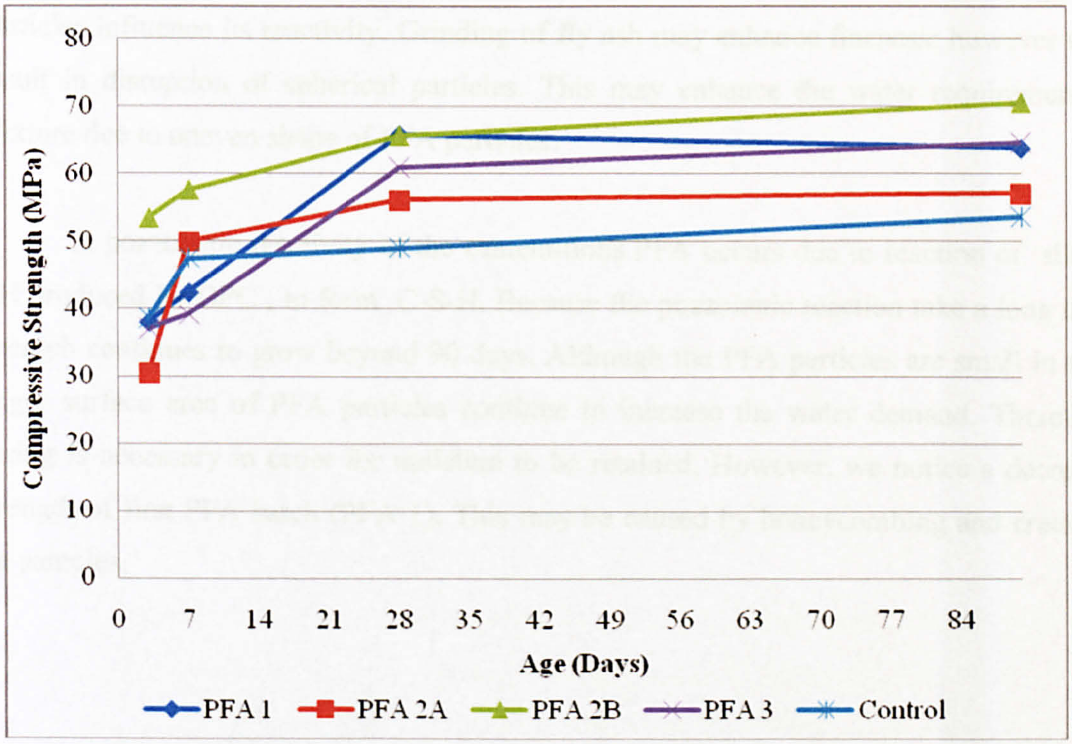

Figure 4.1.1: *Average Compressive strength of PFA versus curing age*

Table 4.1.1 shows the compressive strength test results for PFA mixtures. For all mixtures, the replacement level of cement was kept constant (10% PFA). However, the water-to-cement ratio varied from 0.25 to 0.36. Each value represents the average of three experimental observations. At early ages (3 days), the PFA concrete mixes obtained strength between 30.3MPa and 53.53MPa as compared to 38.53 MPa of control concrete. The strength of PFA-2A specimens was lower as compared to Control mixtures due to segregation. Segregation occurred to incorrect water ratio. Hence, the mixture (PFA 2A) was replaced with PFA 2B which produced a high early strength than all mixtures.

The strength difference between the PFA-concretes and OPC specimens became more evident at 28 days, whereby PFA achieved **25, 5%** higher strength than OPC concrete. Such differential occurs due to the fineness and pozzolanic reactivity of PFA, which is noticeable after 10-14 days of hydration process. Also, the glass content and the spherical shape of PFA particles influence its reactivity. Grinding of fly ash may enhance fineness; however this may result in disruption of spherical particles. This may enhance the water requirement of the mixture due to uneven shape of PFA particles.

A pozzolanic reactivity of the cementitious PFA occurs due to reaction of silica with CH produced by OPC , to form C-S-H. Because the pozzolanic reaction take a long time, the strength continues to grow beyond 90 days. Although the PFA particles are small in size, the larger surface area of PFA particles continue to increase the water demand. Therefore wet curing is necessary in order for moisture to be retained. However, we notice a decrement in strength of first PFA batch (PFA 1). This may be caused by honeycombing and crack within the samples.

Table 4.1.2: Average Compressive strength of MIRHA and PFA versus curing age

Average Compressive Strength, MPa				
Days	**3**	**7**	**28**	**90**
MIRHA + PFA 1	37	48.23	58.12	69.94
MIRHA + PFA 2	37.14	50.01	65.17	68.01
MIRHA + PFA 3	40.95	45.82	59.83	62.61
Control	38.53	47.45	49.02	53.70

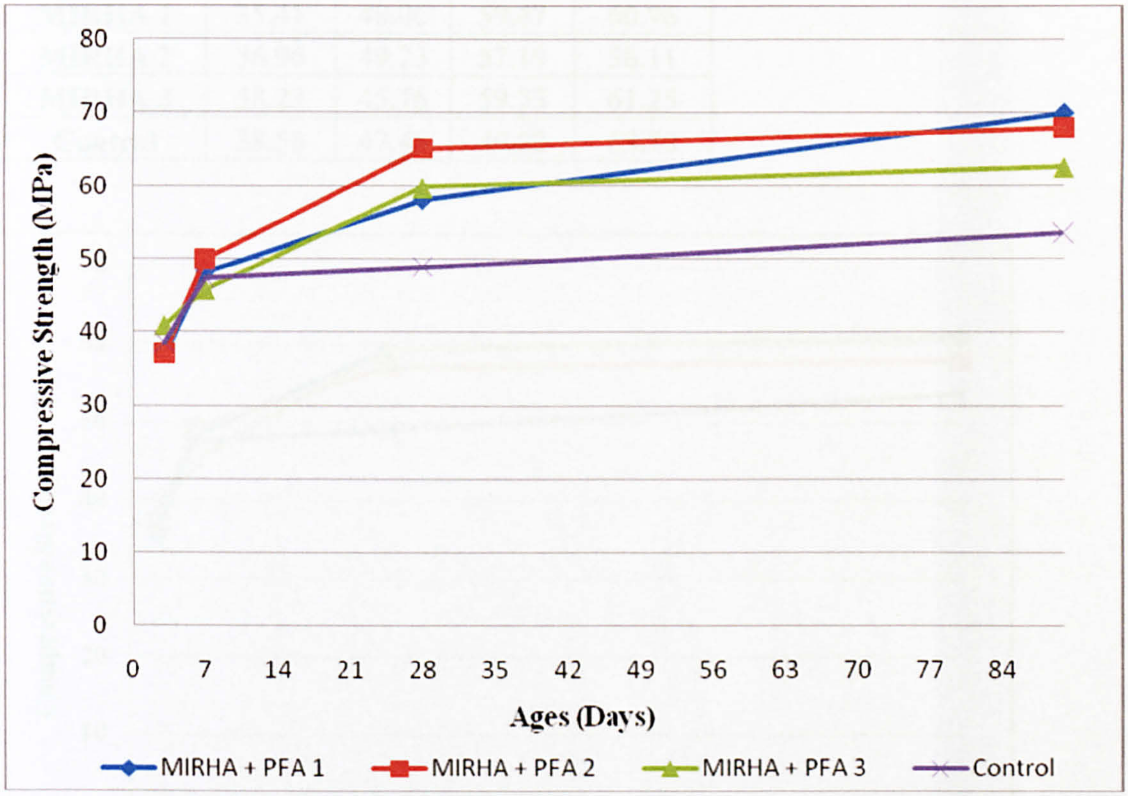

Figure 4.1.2: *Average Compressive strength of MIRHA and PFA versus curing age*

Figure 4.1.2 represents results of Table 4.1.2. For all mixtures, 10% of MIRHA and PFA were incorporated into the concrete. At early stages (3 days), we notice that concrete with OPC only has higher strength than concrete with MIRHA and PFA. Since, PFA and MIRHA have different particle sizes, their surface areas are different. Thus, water demand is not evenly distributed amongst the constituents. Hydration process is an exorthemic reaction which releases heat. Both replacement materials produce different heat evolution, which produces fluctuation in the hydration process. Due to the This phenomenon explain the drastic increase of strength at 7 days.

After 28 days, the first batch of MIRHA and PFA continues to increase drastically; while other batches gradually increase in strength. While MIRHA is responsible for the early strength development, PFA is responsible for the late strength development.

Table 4.1.3: Average Compressive strength of MIRHA versus curing age

Average Compressive Strength, MPa				
Days	**3**	**7**	**28**	**90**
MIRHA 1	35.41	48.06	59.47	60.96
MIRHA 2	36.96	49.23	57.19	58.11
MIRHA 3	38.23	45.16	59.33	61.25
Control	38.58	47.45	49.02	53.70

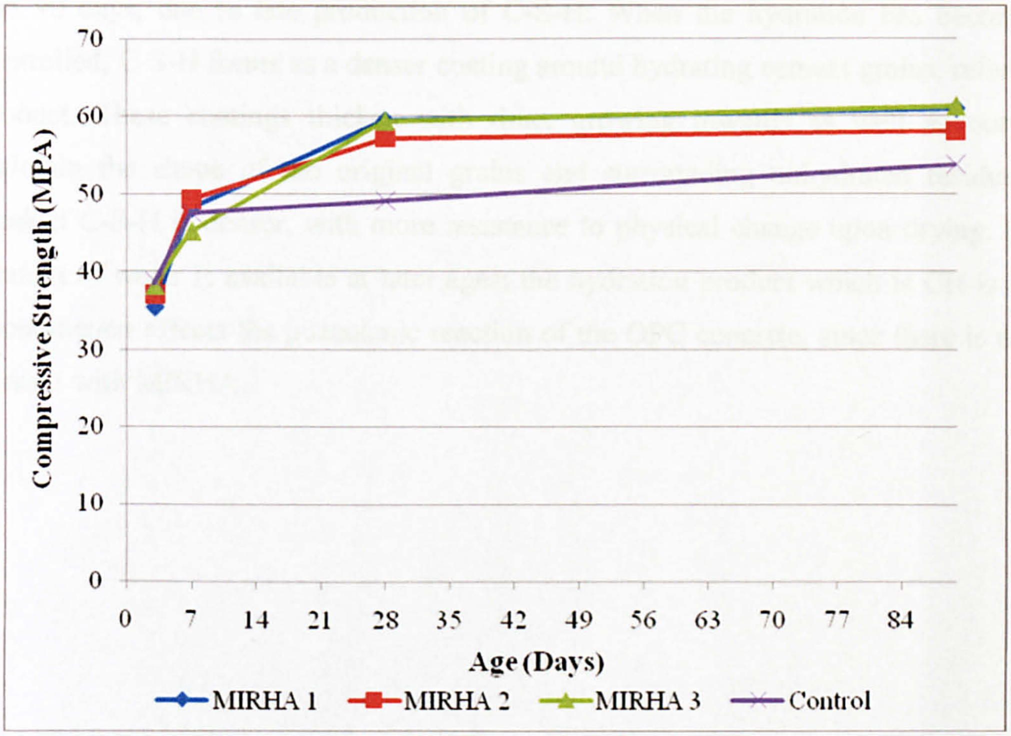

Figure 4.3: *Average Compressive strength of MIRHA versus curing age*

Figure 4.1.3 is a graphical presentation of the results in Table 4.1.3. For all mixtures, 10% of MIRHA was used, whereas the water-to-cement (w/c) ratio varied between 0.39 and 0.47. At early stages, we can see that the strength of MIRHA samples obtained strength less than control-concrete sample. Due to the adsorption character of Cellular MIRHA particles, concrete containing MIRHA require more water for a given consistency.

For a given consistency, the reduction of water requirement can lead an overall improvement in many engineering properties. At 7 days, we notice an increase in strength of both OPC concrete and MIRHA concretes. Due to adsorptive properties of MIRHA particles and large surface area, concrete containing MIRHA tends to absorb more water, meaning less water is available for the hydration of cement. The cement particles react with water to form calcium-silicate-hydrate(C-S-H) and calcium hydroxide (CH). The formation of CH at the surface of MIRHA is due to the adsorption by cellular structure of MIRHA. The adsorbed water enhances the pozzolanic reaction inside the cellular spaces and gain significant strength. Pozzolanic reaction occurs, whereby MIRHA reacts with CH.

A small increase in compressive strength of MIRHA concrete is notable between 28 and 90 days, due to late production of C-S-H. When the hydration has become diffusion-controlled, C-S-H forms as a denser coating around hydrating cement grains, referred to as late product. These coatings thicken with time, growing inwards as well as outwards. They maintain the shape of the original grains and surrounding unhydrated residues. This late product C-S-H is denser, with more resistance to physical change upon drying. Due to small amount of water is available at later ages; the hydration product which is CH is limited. This phenomenon affects the pozzolanic reaction of the OPC concrete, since there is not much CH to react with MIRHA.

4.2 SPLIT-CYLINDER TEST RESULTS

The following equation is used for calculating of horizontal tensile stress.

$$f_t = \frac{2P}{\pi L D}$$

where P = compressive load on the cylinder

L = length of the cylinder, and

D = diameter

Tensile strength of cylinders of each mix was measured at the curing age of 90 days. The results of tensile strength mixes are shown on Figure 4.2.1 below. Generally, the graph shows an increment in the tensile strength of the concrete as the percentage of fly ash increases.

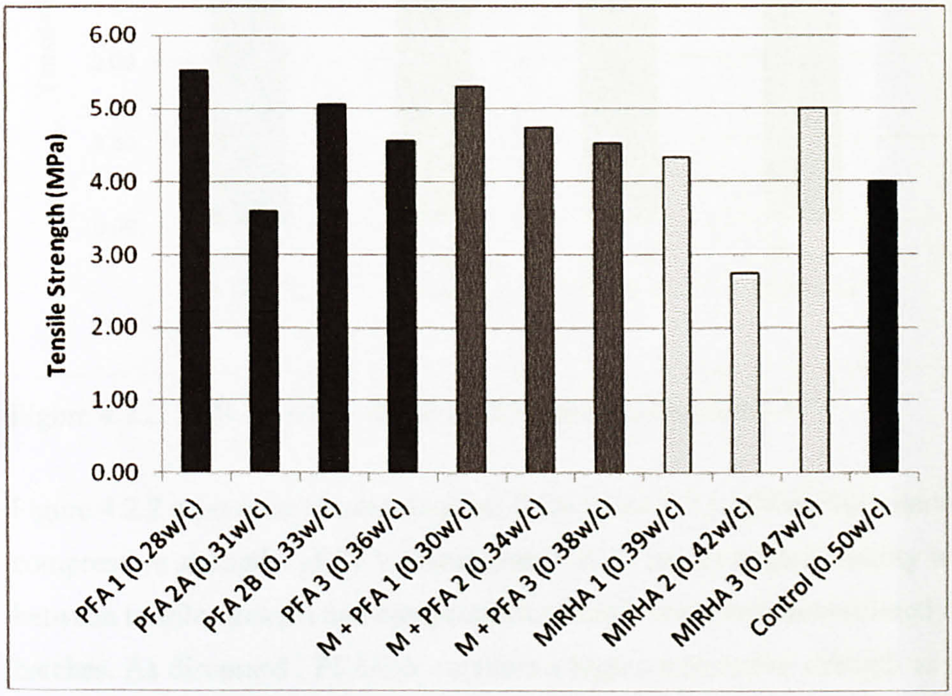

Figure 4.2.1: *Split-tensile strength of SCC*

Generally, the figure shows an increment in the tensile strength, f_t of the concrete as the water/binder ratio decreases. The calculated values of tensile strength, f_t was obtained using above formula where the P is the compressive load of standard test cylinder in Newton.

Table 4.2.1: Average Compressive Strength versus Average tensile strength of PFA

MIX	Compressive Strength (MPa)	Average Tensile Strength (MPa)
PFA 1	63.83	5.52
PFA 2A	57.13	3.60
PFA 2B	70.54	5.05
PFA 3	64.76	4.54
Control	53.70	3.99

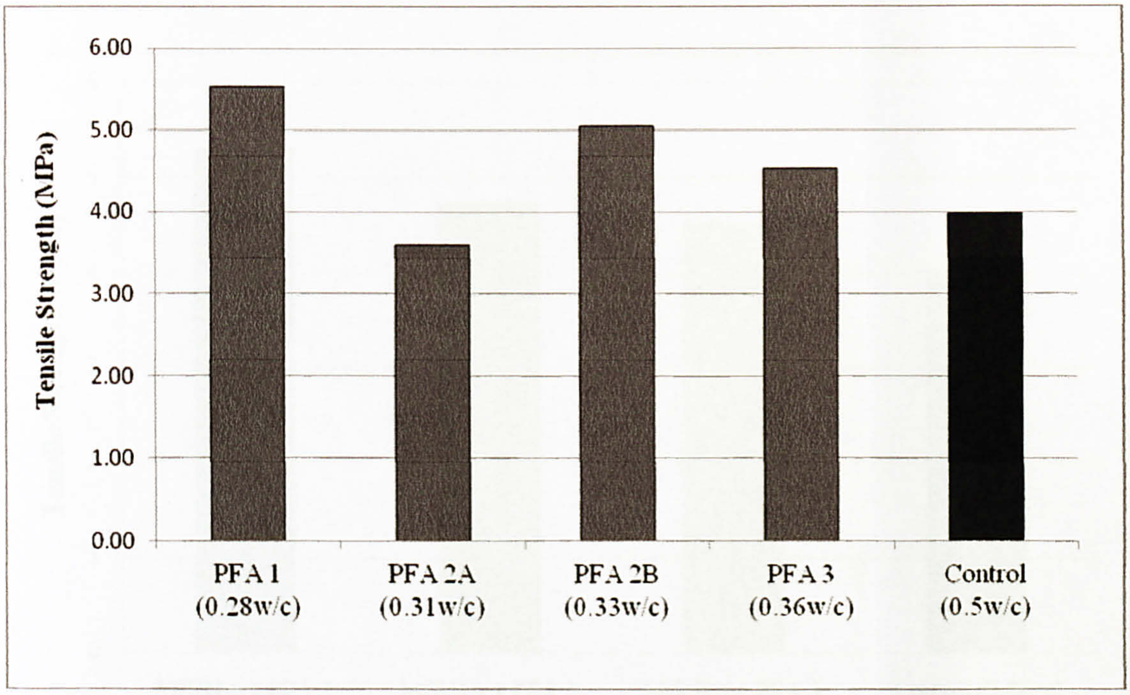

Figure 4.2.2: *Split-tensile strength of SCC incorporated with PFA*

Figure 4.2.2 represents results obtained from Table 4.2.1, giving the tensile strength for various compressive strengths of PFA-incorporated SCC. Direct proportionality was obtained between between tensile strength and compressive strength is not common (directly-proportional) for all batches. As discussed , PFA-2A obtained a high compressive strength as compared to Control mix due to segregation; however the tensile strength is closely related to that of Control mix. We also note that the highest compressive strength does not necessarily produce the highest tensile strength. This proves that there is a close relationship between the splitting-tensile strength and compressive strength. The ratio of the two strength can be influenced by many factors such as age, and grading of coarse aggregate. According to Neville, tensile strength is sensitive to inadequate curing than the compressive strength.

Table 4.2.2: Average Compressive Strength versus Average tensile strength of MIRHA and PFA

MIX	Compressive Strength (MPa)	Average Tensile Strength (MPa)
MIRHA + PFA 1	69.94	5.29
MIRHA + PFA 2	68.01	4.73
MIRHA + PFA 3	62.61	4.51
Control	53.70	3.99

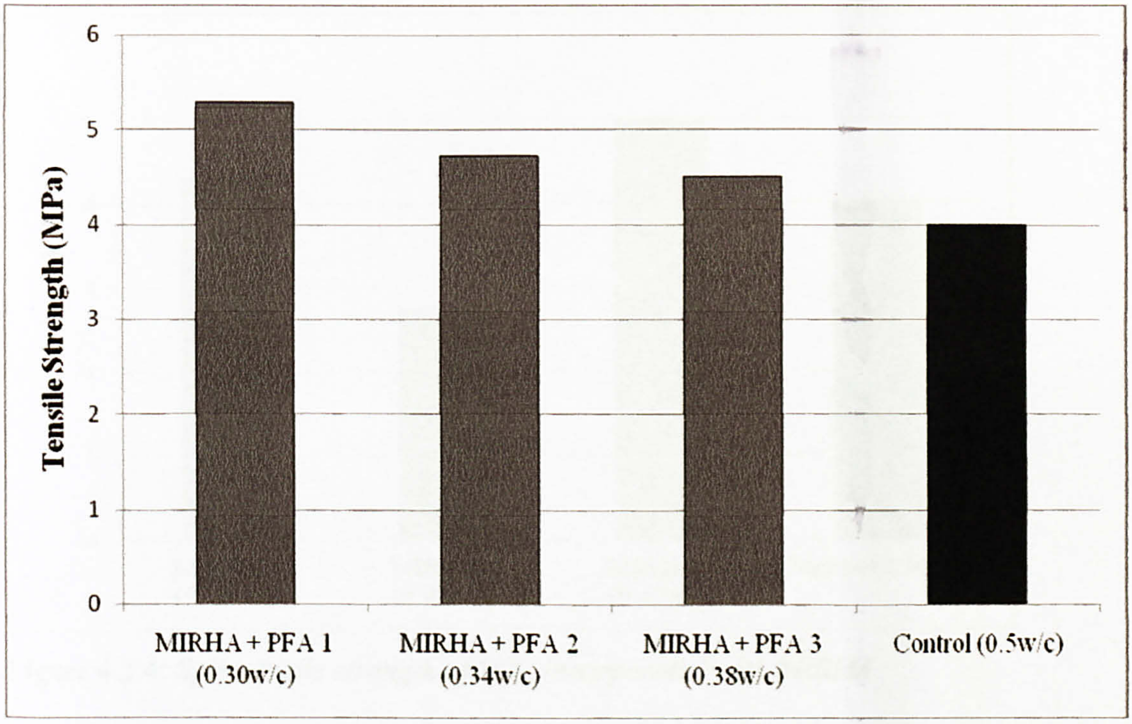

Figure 4.2.3: *Split-tensile strength of SCC incorporated with MIRHA and PFA*

Figure 4.2.3 represents results obtained from Table 4.2.2, giving the tensile strength for various compressive strengths of SCC incorporated with PFA and MIRHA. From the figure, we obtain a linear trend amongst the various strengths. As the compressive strength increases, the splitting tensile strength increases. Thus, the relationship between the two strengths is directly proportional. Furthermore, the tensile strength reduces with respect to an increase in water-to-binder ratio.

Table 4.2.3: Average Compressive Strength versus Average tensile strength of MIRHA

MIX	Compressive Strength (MPa)	Average Tensile Strength (MPa)
MIRHA 1	60.96	4.32
MIRHA 2	58.11	2.75
MIRHA 3	61.25	4.99
CONTROL	53.7	3.99

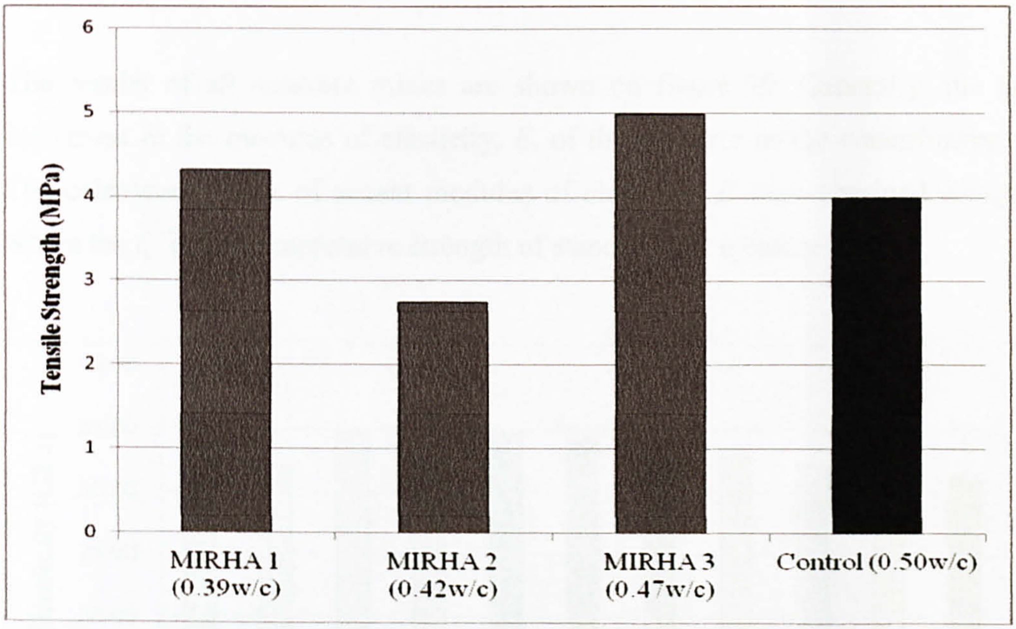

Figure 4.2.4: *Split-tensile strength of SCC incorporated with MIRHA*

Figure 4.2.4 represents the results from Table 4.2.3, giving tensile strength for various compressive strengths of SCC incorporated with MIRHA. From the figure, there is relationship between the tensile strength and water-to binder ratio. Notably, the tensile strengths for SCC incorporated with MIRHA, are lower than that of PFA-incorporated SCC with similar compressive strength.

4.3 PRISM TEST RESULTS

The expression for secant modulus of elasticity of concrete, E_c, as recommended by ACI 318-89 for structural calculation of normal weight concrete with strength up to 83MPa is:

$$E = 3.32(f_c')^{0.5} + 6.9$$

where E_c = Secant of Modulus Elasticity (GPa)

f_c' = compressive strength of standard test cylinder in MPa.

The results of all concrete mixes are shown on figure 29. Generally, the graph shows an increment in the modulus of elasticity, *E*, of the concrete as the water/binder ratio decreases. The calculated values of secant modulus of elasticity, *E* were obtained using above formula where the f_c' is the compressive strength of standard test cylinder in MPa.

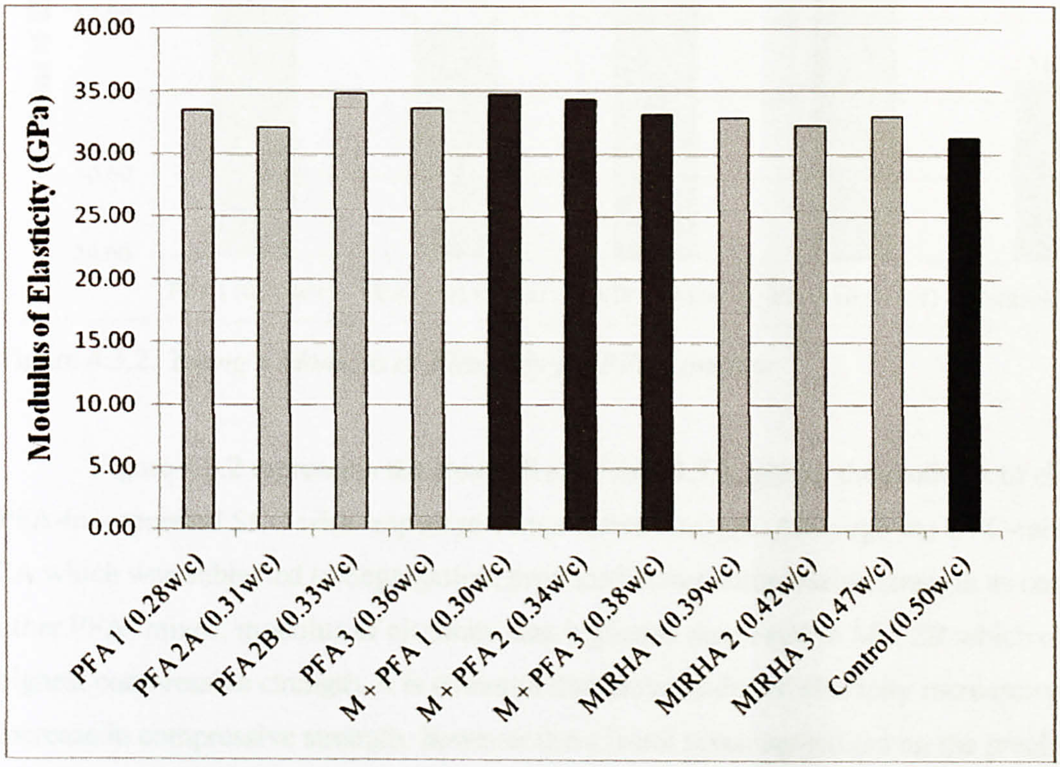

Figure 4.3.1: *Modulus of Elasticity for SCC*

Table 4.3.1: Young's Modulus of Elasticity versus average Compressive Strength of PFA

MIX	Compressive Strength (MPa)	Young's Modulus (GPa)
PFA 1	63.83	33.40
PFA 2A	57.13	31.99
PFA 2B	70.54	34.78
PFA 3	64.76	33.62
Control	53.70	31.23

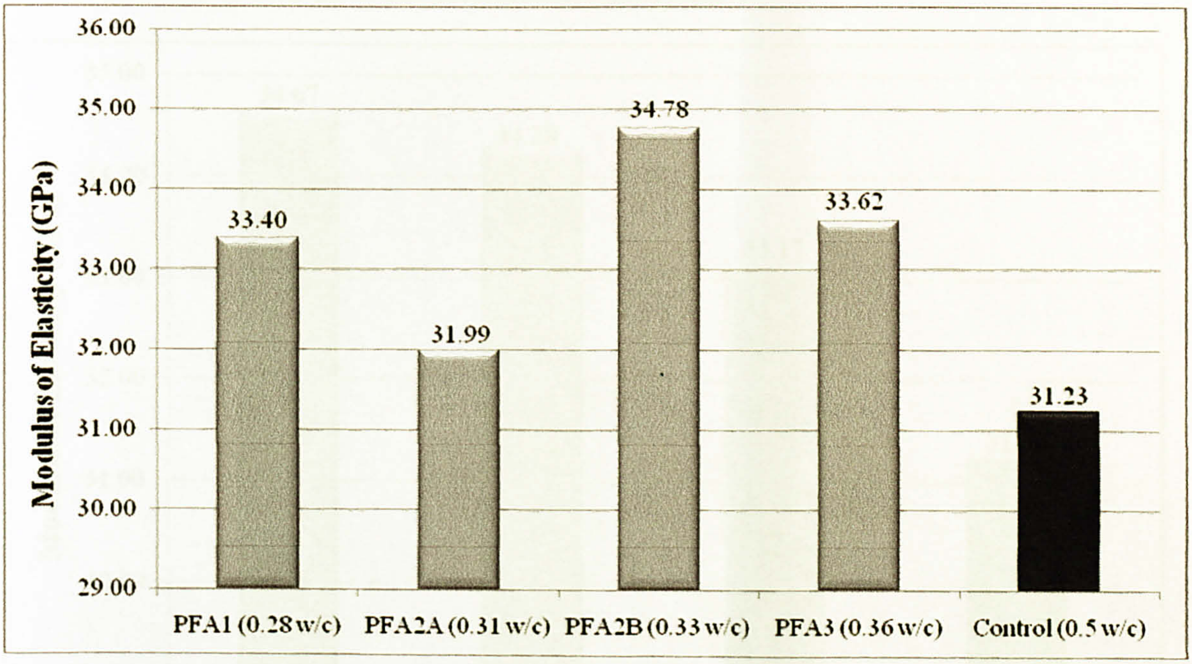

Figure 4.3.2: *Young's Modulus of Elasticity for PFA-concrete*

Figure 4.3.2 represents the results from Table 4.3.1, giving the modulus of elasticity of PFA-incorporated SCC with respect to compressive strength. Although the OPC-mix, and Mix 2A which was subjected to Segregation, produced lower compressive strength as compared to other PFA- mixes; modulus of elasticity was higher as compared to Mix 2B which obtained the highest compressive strength. It is common that the modulus of elasticity increases with an increase in compressive strength, however there is not given agreement on the precise relationship of the two quantities. Instead, the modulus of elasticity of concrete is influenced by the modulus of elasticity of aggregate and the volumetric proportion of aggregate in concrete. Since the obtained values are not from the same mix, the influence of water-to-binder ratio plays a significant role.

Table 4.3.2: Young's Modulus of Elasticity versus average Compressive Strength of MIRHA and PFA

MIX	Compressive Strength (MPa)	Young's Modulus (GPa)
MIRHA + PFA 1	69.94	34.67
MIRHA + PFA 2	68.01	34.28
MIRHA + PFA 3	62.61	33.17
Control	53.70	31.23

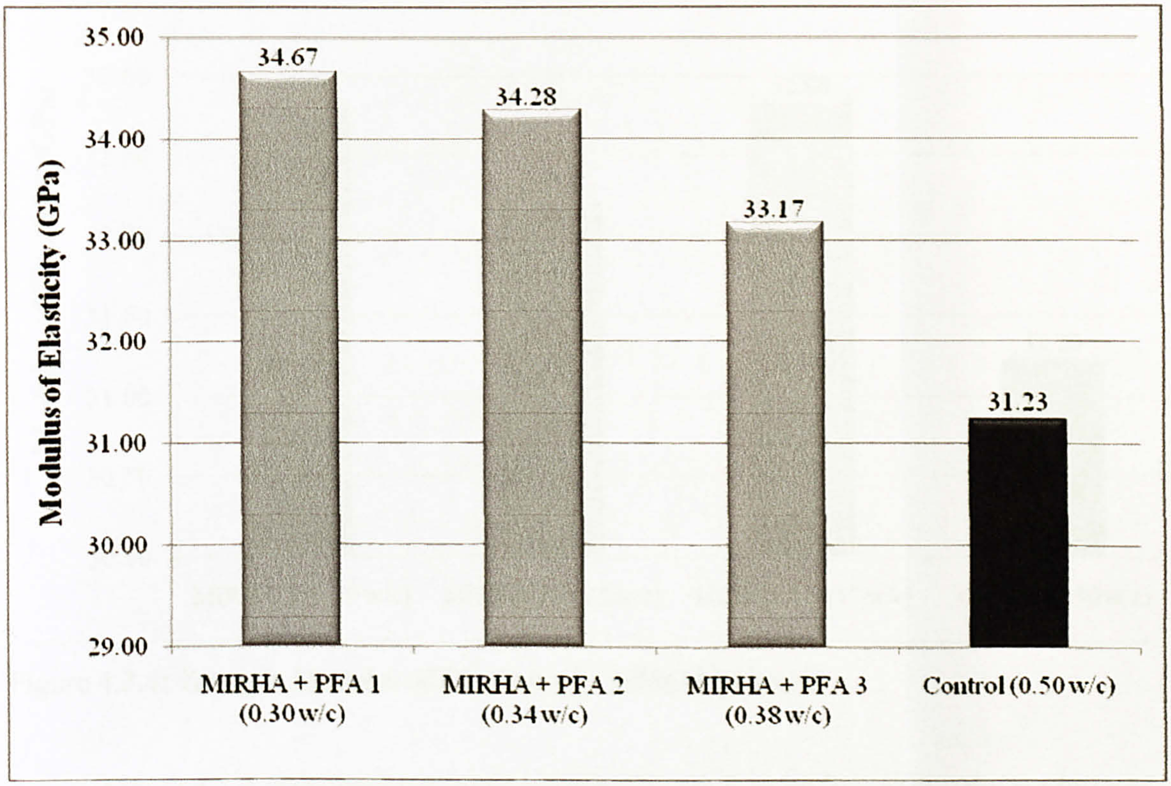

Figure 4.3.3: *Young's Modulus of Elasticity for PFA and MIRHA concrete*

Figure 4.3.3 represents the results from Table 4.3.2, giving the modulus of elasticity for SCC incorporated with MIRHA and PFA. We obtained higher moduli of elasticity of SCC with MIRHA and PFA than OPC-mix. The values obtained, corresponds to higher compressive strengths of respective mixes. The values of moduli can be affected by the performance of the hydrated cement. As discussed, water requirements for SCC incorporated with both MIRHA and RHA is not evenly distributed with every batch due to their various heat evolution during hydration.

Table 4.3.3: Young's Modulus of Elasticity versus average Compressive Strength of MIRHA

MIX	Compressive Strength (MPa)	Young's Modulus (MPa)
MIRHA 1	60.96	32.82
MIRHA 2	58.11	32.21
MIRHA 3	61.25	32.88
CONTROL	53.7	31.23

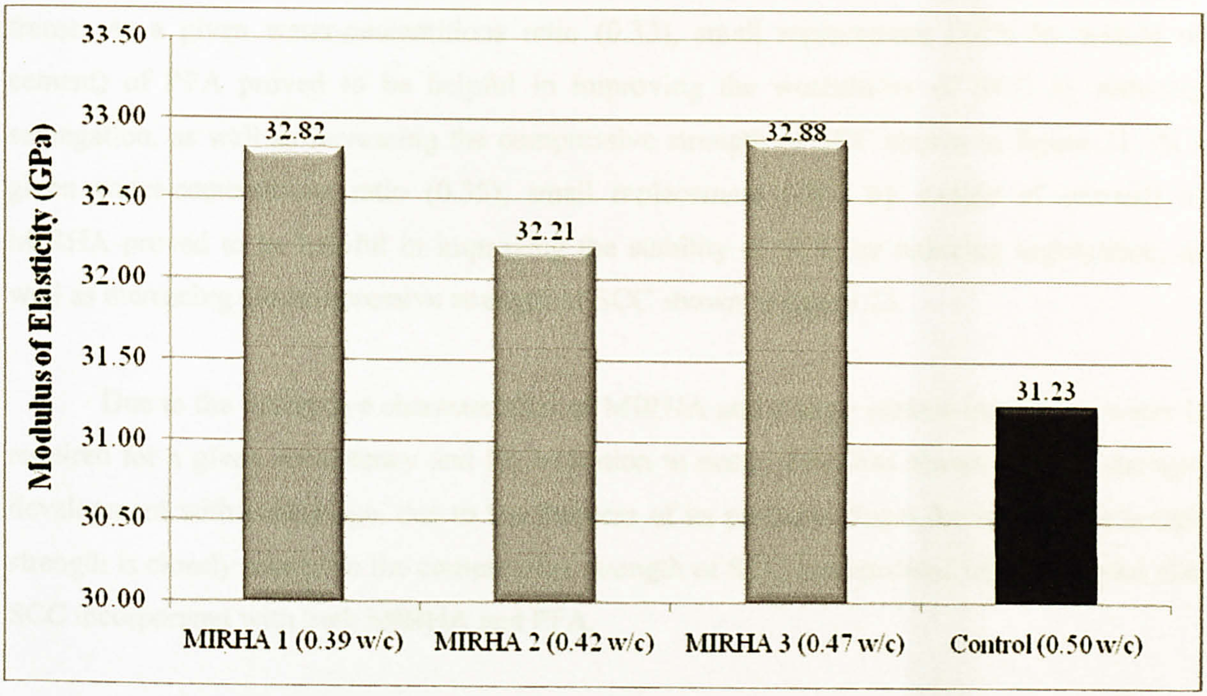

Figure 4.3.4: *Young's Modulus of Elasticity for MIRHA concrete*

Figure 4.3.4 represents the results obtained from Table 4.3.3, giving the modulus of elasticity for MIRHA-incorporated SCC. From the figure, a decrement in modulus of elasticity,E of concrete is with respect to higher water-to-cement ratio. However, the E-value of MIRHA as compared to OPC-mix, is higher. This corresponds to the higher compressive strength for the MIRHA-incorporated SCC as compared to OPC concrete.

CHAPTER 5
CONCLUSION AND RECOMMENDATIONS

5.1 Conclusion

The objectives of this project, which is to determine the tensile strength and modulus of elasticity of SCC made from MIRHA and PFA; and to determine the effects of PFA and MIRHA on the compressive strength of SCC, were successfully achieved within the time frame. At a given water-cementitious ratio (0.33), small replacement (10% by weight of cement) of PFA proved to be helpful in improving the workability of SCC by reducing segregation, as well as increasing the compressive strength of SCC shown in figure 21. At a given water-cementitious ratio (0.35), small replacement (10% by weight of cement) of MIRHA proved to be helpful in improving the stability of SCC by reducing segregation, as well as increasing the compressive strength of SCC shown in figure 23.

Due to the adsorptive characteristics of MIRHA and a large surface area, more water is required for a given consistency and for hydration to occur. PFA has shown a better strength development with curing age, due to the fineness of its particles. From the results, the tensile strength is closely related to the compressive strength of SCC incorporated with PFA, and also SCC incorporated with both MIRHA and PFA.

Concrete incorporated with PFA produced 23.6% and 23.5% higher compressive strength at 28 and 90 days respectively, than OPC concrete. Concrete incorporated with PFA and MIRHA produced 21% and 26.6% higher compressive strength at 28 and 90 days respectively, than OPC concrete. Concrete incorporated with MIRHA produced 17.7% and 18.5% higher compressive strength at 28 and 90 days respectively, than OPC concrete. The findings suggest that an incorporation of PFA and MIRHA is better than that of MIRHA only.

With respect to the results of related hardened properties of concrete mixes studied, it can be summarized that the use of PFA and MIRHA as cement replacement in concrete mix, together with variable water-binder ratio (w/b) had exhibited significant results and effects in the values studied, which is the modulus of elasticity and tensile strength of SCC.

5.2 Recommendations

In order to improve the reliability of the results and to obtain further research regarding the topic, the following recommendations should be highlighted:

- Continue the project with different percentages of PFA and MIRHA as cement replacement materials to further see the effect of addition to the SCC.
- The time frame for this project should be longer in order to compare results of older ages than 90 days.
- Since pozzolanic reaction is a slow process, the samples can be cured under steam curing, which will speed up the pozzolanic reaction in a short time.

CHAPTER
6
REFERENCES

1. Satish Chandra, *Waste Materials Used in Concrete Manufacturing*, Noyes Publication, 1997 *(pp 175 – 230)*

2. S Popovics, *Strength and Related Properties of Concrete*, John Wiley & Sons, Inc. 1998

3. *High Performance Concrete*, P. –C. Aïtcin, 1998

4. Klieger P. and Lamond J, 1994, "*Significance of tests and Properties of Concrete and Concrete-Making Materials*, 4th Edition, ASTM STP 169C.

5. M Khrapko, (1999) *Self Compacting Concrete: Solution for Technology Hungry, Concrete Construction.*

6. Bartos PJM, Zhu W and Quinn J (2002), *Aspects of durability of SCC*, University of Paisley, Scotland, UK

7. A.M Neville, *Properties of Concrete*, Addison Wesley Longman Ltd. , 1995

8. G De Schutte, PJM Bartoz, P Domone and J Gibbs, *Self-Compacting Concrete*, Whittles Publishing, 2008 (pp. 166-192).

9. RK Dhir, TG Jappy, "*Exploiting wastes in concrete*", Thomas Telfold Publishing, 1995, (pp 283 -289)

10. I K Cisse, R Jauberthie, M Temimi and JP Camps, Rice hush ash: A *filler for sand concrete. Proceedings of Exploiting wastes in concrete, Creating with concrete*, Dundee 1999, (pp 308-317)

11. R. Siddique, "Waste materials and by-products in concrete", Springer-Verlag, Berlin, 2008 (pp 235 -263)

12. C.L Page, MM Page, "*Durability of concrete and cement composites", Woodhead Publishing, 2007*

13. J Newman, B Seng Choo, "*Advance Concrete Technology: Concrete properties*", Elsevier Butterworth Heinemann, 2003

14. J Newman, B Seng Choo, "*Advance Concrete Technology: Tests and quality*", Elsevier Butterworth Heinemann, 2003

15. J Newman, B Seng Choo, "*Advance Concrete Technology: Constituents Materials*", Elsevier Butterworth Heinemann, 2003

16. J Newman, B Seng Choo, "*Advance Concrete Technology: Processes*", Elsevier Butterworth Heinemann, 2003

17. M.I. Khan, C.J. Lynsdale, P. Waldron, 2000, "*Porosity and strength of PFA/SF/OPC ternary blended paste",* Centre for Cement and Concrete, Department of Civil and Structural Engineering, University of Sheffield

18. M. Tia, Y Liu, D Brown, 2002, "Modulus of elasticity, creep and shrinkage of concrete", Department of Civil & Coastal Engineering, University of Florida.

19. H.J.H. Brouwers, H.J. Radix, 2004, "*Self-Compacting Concrete: Theoretical and experimental study*", Department of Civil Engineering, Faculty of Engineering Technology, University of Twente, Netherlands.

20. B. Persson, 2000, "*A comparison between mechanical properties of self-compacting*

21. M Sonebi, 2003, "*Medium strength self-compacting concrete containing fly ash:*

22. *Modelling using factorial experimental plans*", Advanced Concrete and Masonry Centre, University of Paisley, Paisley, UK.

23. G. De Schutter, 2005, "Guidelines for testing fresh self-compacting concrete, UK.

24. B Felekoglu, S. Tűrkel, B. Baradan, 2006, *"Effect of water/cement ratio on the fresh and hardened properties of self-compacting concrete"*, Department of Civil Engineering, Dokuz Eylul University, Izmir, Turkey

25. C. Roccoa, G.V. Guineab, J. Planasb, M. Elices, 2000, *"Review of the splitting-test standards from a fracture mechanics point of view"*, Universidad Nacional de la Plata, La Plata, Argentina

26. M. A. Ahmadi, O. Alidoust, I. Sadrinejad, and M. Nayeri, 2007, *"Development of Mechanical Properties of Self-Compacting Concrete Contain Rice Husk Ash"*, World Academy of Science

27. F. Tomosawa and T.Noguchi, *"Relationship between compressive strength and modulus of elasticity of high-strength concrete"*, Department of Architecture, Faculty of Engineering, University of Tokyo

APPENDICES

1. COMPRESSIVE STRENGTH

Table 4.1.1: Average Compressive strength of PFA versus curing age

Average Compressive Strength, MPa				
Day	**3**	**7**	**28**	**90**
PFA 1	38.01	42.43	65.78	63.83
PFA 2A	30.34	49.93	56.14	57.13
PFA 2B	53.53	57.65	65.63	70.54
PFA 3	37.06	39.14	61.02	64.76
Control	38.53	47.45	49.02	53.70

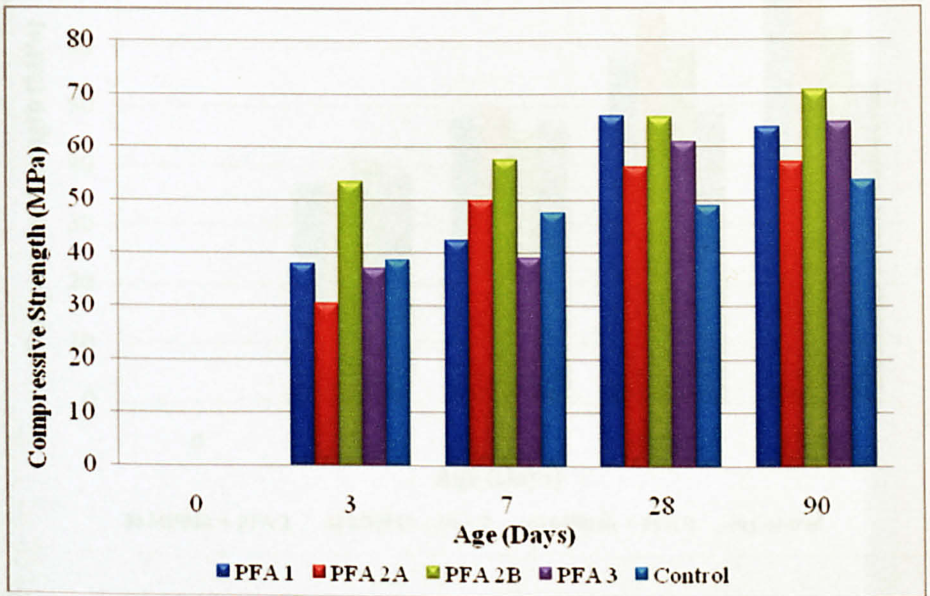

Figure 1.1: *Average Compressive strength of PFA versus curing age*

Table 4.1.2: Average Compressive strength of MIRHA and PFA versus curing age

Average Compressive Strength, MPa				
Days	**3**	**7**	**28**	**90**
MIRHA + PFA 1	37	48.23	58.12	69.94
MIRHA + PFA 2	37.14	50.01	65.17	68.01
MIRHA + PFA 3	40.95	45.82	59.83	62.61
Control	38.53	47.45	49.02	53.70

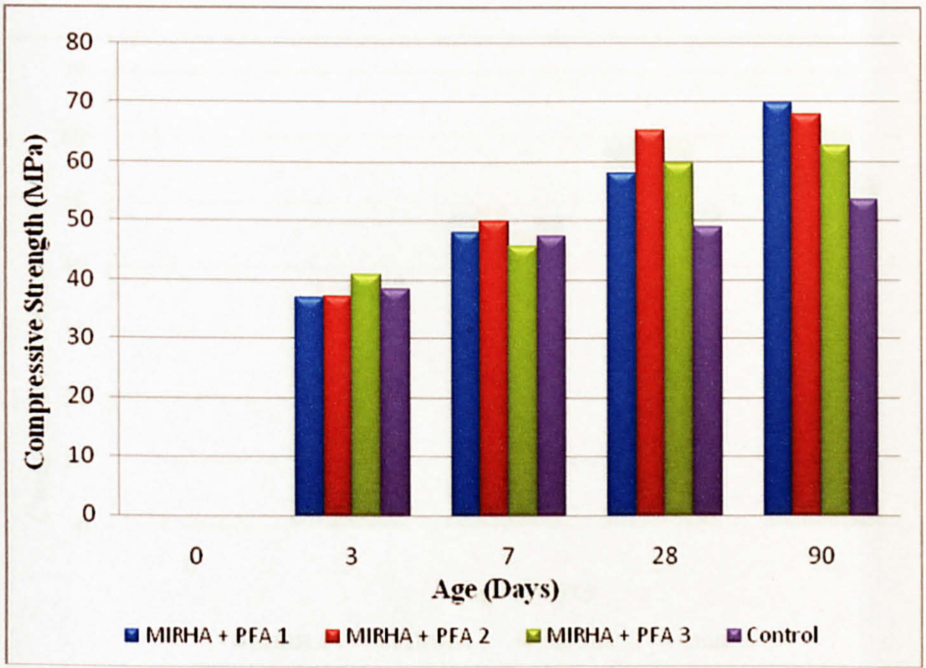

Figure 1.2: *Average Compressive strength of MIRHA and PFA versus curing age*

Table 4.1.3: Average Compressive strength of MIRHA versus curing age

Average Compressive Strength, MPa				
Days	**3**	**7**	**28**	**90**
MIRHA 1	35.41	48.06	59.47	60.96
MIRHA 2	36.96	49.23	57.19	58.11
MIRHA 3	38.23	45.16	59.33	61.25
Control	38.58	47.45	49.02	53.70

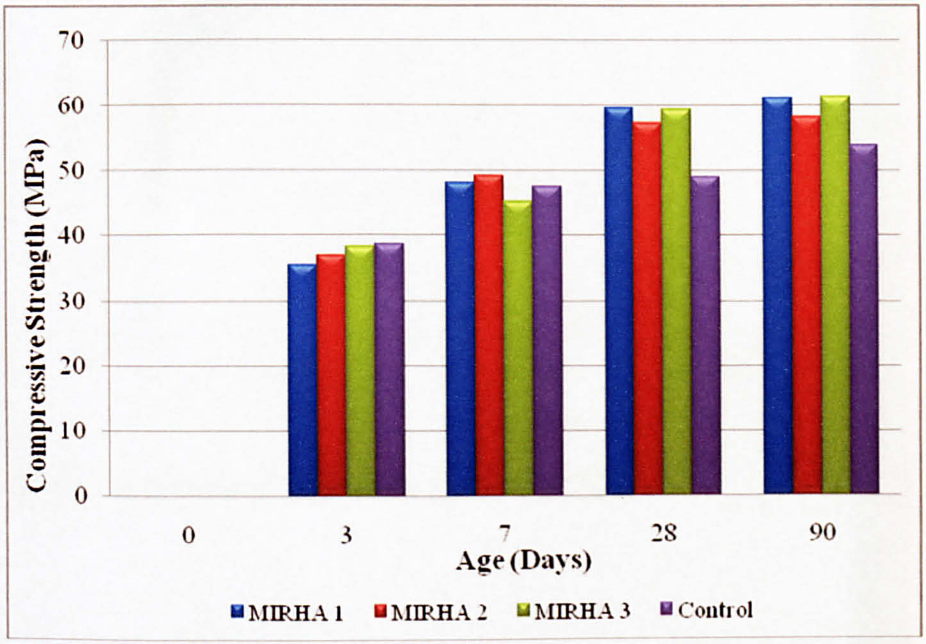

Figure 1.3: *Average Compressive strength of MIRHA versus curing age*

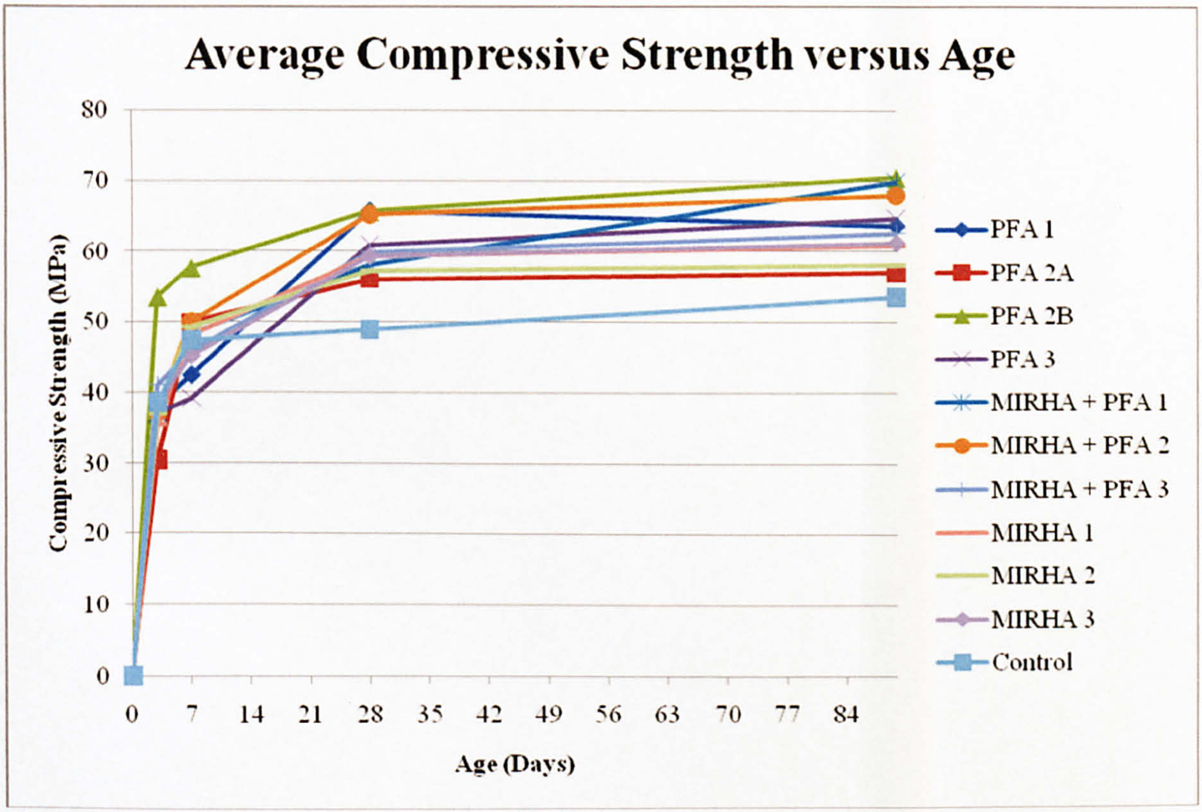

Figure1.4: *Average Compressive Strength of SCC*