THE EFFECT OF AGGREGATE AND MINERAL ADMIXTURES ON ENGINEERING PROPERTIES OF HIGH STRENGTH SELF COMPACTING CONCRETE

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THESIS SUBMITTED IN FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE OF DOCTOR OF PHILOSOPHY

INSTITUTE OF GRADUATE STUDIES
UNIVERSITY OF MALAYA
KUALA LUMPUR

2013

UNIVERSITI MALAYA

ORIGINAL LITERARY WORK DECLARATION

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Registration/Matric No: **KHA080010**

Name of Degree: **Doctor of Philosophy**

Title of Project Paper/Research Report/Dissertation/Thesis ("this Work"): **The Effect of Aggregate and Mineral Admixtures on Engineering Properties of High Strength Self Compacting Concrete**

Field of Study: **Concrete Technology**

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ABSTRACT

This study deals with producing high strength self compacting concrete (HSSCC) using locally available materials. The work was conducted in three parts i.e. paste, mortar and concrete to facilitate the mix design process. The effect of using powder materials and superplasticizer on properties of paste, mortar and concrete as well as the effect of coarse aggregate properties namely volume, maximum size, grading, and shape on the properties of concrete were investigated.

For the study on mortar, fly ash (FA) and metakaolin (MK) as pozzolan while limestone powder and kaolin as filler material were used at replacement levels of 5%, 10%, 15%, and 20% by weight of cement and sand, respectively. Self compactibility of mortars was obtained by adding suitable materials such as mineral admixtures and superplasticizer which provided a sufficient balance between flowability and viscosity of the mix. The optimum replacement level of FA and MK for cement was 10% from the viewpoint of workability and strength, while the optimum replacement level of limestone for sand was 20%. Flowability of mortar increased with the use of fly ash and decreased with the use of metakaolin and kaolin. Moreover, strength of mortar increased when the optimum replacement level of pozzolan and limestone powder was used.

Different fresh concrete tests were adopted. Slump flow spread and V-funnel tests were used to determine the filling ability, L-box and J-ring tests were used to determine passing ability and segregation index test to determine segregation resistance. The results obtained for fresh concrete properties showed that flowability of concrete increased with increase flowability of mortar. The mixes which contained coarse aggregate with higher volume, larger size, non continuous grading and high flakiness ratio affected negatively the fresh properties of HSSCC. The results for the hardened concrete showed better strengths and stiffness as well as good durability for limestone powder, metakaolin, fly ash mixes in the respective order. Higher shrinkage was obtained for the limestone, fly ash mixes in the respective order, while control and metakaolin mixes showed lower shrinkage. The 33% coarse aggregate volume in the mixes was the optimum volume from the viewpoint of strengths and stiffness compared to other volumes. The increase in aggregate volume reduced the shrinkage. Moreover, the strengths and stiffness increased for mixes containing coarse aggregate with smaller particle fractions, continuous grading and low flakiness ratio. Drying shrinkage of the mixes decreased when the larger particles were used while the use of non graded aggregate and flaky particles led to increase in shrinkage. This study concludes that the use of powders gave good workability and strength and thus would reduce the use chemical admixtures and cement and consequently reduce the cost. The flowability of mortar has a great role in affecting the flowability of concrete. Good correlations were found between the results of the fresh and hardened tests for HSSCC, and an equation is proposed to estimate the coarse aggregate volume for HSSCC using the flowability of mortar. It is feasible to produce HSSCC with different powders and different aggregate properties.

ABSTRAK

Kajian ini dijalankan adalah untuk menghasilkan konkrit terpadat sendiri berkekuatan tinggi dengan menggunakan bahan tempatan sedia ada. Kajian dilaksanakan dalam tiga peringkat iaitu pes, mortar dan konkrit untuk menyeimbangkan proses rekabentuk campuran. Kesan penggunan bahan berbentuk serbuk dan 'superplasticizer' pada sifat pes, mortar dan konkrit sebagaimana kesan sifat batu baur kasar seperti isipadu, saiz maksimum, pengredan, betuk dan sifat konkrit itu diselidik.

Bagi kajian terhadap mortar, serbuk abu terbang (FA), dan metakaolin (MK) bertindak sebagai bahan pozzolan manakala serbuk batu kapur dan kaolin bertindak sebagai bahan pemenuh. Sebanyak 5%, 10%, 15% dan 20% mineral dijadikan bahan ganti daripada berat simen dan pasir, serta sebaliknya. Kepadatan sendiri mortar didapati dengan menambah bahan yang bersesuaian seperti bahan tambah mineral dan 'superplasticizer' di mana ia dapat menghasilkan keseimbangan antaranya kebolehaliran dan kelikatan campuran itu. Nilai paras penggantian optimum untuk FA dan MK terhadap simen ialah 10% daripada sudut kebolehkerjaan dan kekuatan, manakala nilai paras penggantian untuk batu kapur sebagai pasir adalah 20%. Kebolehaliran mortar meningkat dengan penggunan serbuk abu terbang dan menurun dengan penggunaan metakaolin dan kaolin. Selain itu juga, kekuatan pada mortar meningkat apabila paras penggantian optimum pada pozzolan dan serbuk batu kapur digunakan.

Ujikaji yang berbeza dilakukan terhadap sifat segar konkrit. Taburan kemerosotan aliran, 'V-funnel' dijalankan untuk menentukan keboleh-penuhan. 'L-box' dan 'J-Ring' dijalankan untuk menentukan kebolehlepasan dan ujian indeks pengasingan untuk

menentukan rintangan pengasingan. Keputusan yang diperolehi daripada sifat segar konkrit menunjukkan kebolehaliran konkrit meningkat dengan meningkatnya kebolehaliran mortar. Campuran yang mengandungi batu baur kasar dengan isipadu yang tinggi, saiz yang besar, metakaolin dan campuran serbuk abu terbang dengan turutan yang betul. Sebaliknya, pengecutan yang tinggi didapati daripada batukapur, campuran serbuk abu terbang dengan turutan yang betul. Campuran kawalan ('control') dan metakaolin menunjukkan pengecutan yang rendah. Peningkatan dalam isipadu batu baur mengurangkan kemungkinan untuk berlakunya pengecutan. Lebih daripada itu, kekuatan dan kekakuan meningkat untuk campuran yang mengandungi batu baur kasar dengan pecahan partikel lebih kecil, penggredan yang berterusan dan rendah nisbah serpihan. Pengecutan kering campuran berkurangan apabila partikel yang besar digunakan manakalan penggunaan batu baur yang tidak digredkan dan partikel yang menggelupas menyebabkan peningkatan pengecutan. Kajian ini merumuskan bahawa penggunan bahan dalam bentuk serbuk memberi kebolehkerjaan yang baik dan kekuatan yang tinggi, dan dapat mengurangkan penggunaan bahan tambah kimia dan simen, sekaligus dapat menjimatkan kos. Korelasi yang baik dihasilkan daripada keputusan ujikaji segar dan ujikaji keras untuk Konkrit Terpadat Sendiri berkekuatan Tinggi (HSSCC). Persamaan terbitan juga dicadangkan untuk menganggar isipadu batu baur kasar untuk HSSCC menggunakan kebolehaliran mortar. Adalah lebih mudah untuk menghasilkan HSSCC menggunakan bahan berbentuk serbuk yang berbeza dan sifat batu baut yang berbeza.

ACKNOWLEDGMENT

In the name of Allah, the author praises and thanks **Allah** Almighty who gave him the capability and patience to finalize this research.

The author would like to introduce sincere gratitude to his dear supervisor Prof. Dr. Hashim Abdul Razak for his support, assistance, and valuable guidance throughout the research. His valuable opinions have been so useful for completing this work.

The author also would like to thank University of Malaya for the provided financial support through IPPP fund (PS119/2008C) and the research grant (UM.C/625/1/HIR/105).

The author also expresses thank to the staff of civil department of the faculty of engineering for their assistance to finalize this work. They do not hesitate in providing whatever good to facilitate the requirement of research.

The author also wishes to express his deepest thank to the supreme people in his life, his parents, wife, brother, sister and cousins, for their continued support and patience during the period of the study.

The author also wishes to introduce his thanks to all of his friends especially (Dr. Moatasem and, Haj Mohamad, Ayad, Ghassan, Ferdi, and Samsul) for their help and encouragement during the period of study. Their valuable guidance, comments, advices and suggestions were so beneficial during the period of the study.

The author
2013

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List of Symbols and Abbreviations

A: Absorption value

ACI: American Concrete Institute

ASTM: The American society of testing and materials

BR: blocking ratio

C Agg: coarse aggregate

C: control mortar

C_3A: Tricalcium aluminate

C_3S: Tricalcium silicate

Ca $(OH)_2$: calcium hydroxide

CBI: Swedish Cement and Concrete Research Institute

CC14: Control mix containing continuous grade with maximum size 14

CC20: Control mix containing continuous grade with maximum size 20

CG20: Control mix containing gap grade with maximum size 20

CH: calcium hydroxide

CIP: concrete in practice

CS10: Control mix containing single size grade with maximum size 10

CS10HFR: Control mix containing single size grade with maximum size 10 with high flakiness ratio

CS14: Control mix containing single size grade with maximum size 14

C-S-H: Calcium silicate hydrate

Ɛ: Strain

$Ɛ_a$:   the mean strain under the upper loading stress;

ε_b: the mean strain under the basic stress.

E_c: modulus of elasticity of concrete

EFNARC: European Federation of National Associations Representing for Concrete

FA: fly ash

FA10: fly ash mortar, 10% replacement level with cement

FA10C14: fly ash mix of 10% cement replaced and containing continuous grade with maximum size 14

FA10C20: fly ash mix of 10% cement replaced and containing continuous grade with maximum size 20

FA10G20: fly ash mix of 10% cement replaced and containing gap grade with maximum size 20

FA10S10: fly ash mix of 10% cement replaced and containing single size grade with maximum size 10

FA10S10HFR: fly ash mix of 10% cement replaced and containing single size grade with maximum size 10 with high flakiness ratio

FA10S14: fly ash mix of 10% cement replaced and containing single size grade with maximum size 14

FA15: fly ash mortar, 15% replacement level with cement

FA20: fly ash mortar, 20% replacement level with cement

FA5: fly ash mortar, 5% replacement level with cement.

FAgg.: fine aggregate

F_{cu}: compressive strength of cube

F_{cy}: compressive strength of cylinder

f_t: flow time of concrete through V-funnel box orifice

f_{t5min}: flow time of concrete through V-funnel box orifice after 5 min refilling

HRWR: high range water reducer

HSC: high strength concrete

HSSCC: high strength self compacting concrete

HSSCM: high strength self compacting mortar

ITZ: interfacial transition zone

JSCE: Japanese Society of Civil Engineers

K: Kaolin

K10: kaolin mortar, 10% replacement level with sand

K15: kaolin mortar, 15% replacement level with sand

K20: kaolin mortar, 20% replacement level with sand

K5: kaolin mortar, 5% replacement level with sand

LOI: Loss on ignition

LP: limestone powder

LP10: limestone powder mortar, 10% replacement level with sand

LP15: limestone powder mortar, 15% replacement level with sand

LP20: limestone powder mortar, 20% replacement level with sand

LP20C14: limestone powder mix of 20% sand replaced containing continuous grade with maximum size 14

LP20C20: limestone powder mix of 20% sand replaced containing continuous grade with maximum size 20

LP20G20: limestone powder mix of 20% sand replaced containing gap grade with maximum size 20

LP20S10: limestone powder mix of 20% sand replaced containing single size grade with maximum size 10

LP20S14: limestone powder mix of 20% sand replaced containing single size grade with maximum size 14

LP5: limestone powder mortar, 5% replacement level with sand

MK: metakaolin

MK10: metakaolin mortar, 10% replacement level with cement

MK10C14: metakaolin mix of 10% cement replaced and containing continuous grade with maximum size 14

MK10C20: metakaolin mix of 10% cement replaced and containing continuous grade with maximum size 20

MK10G20: metakaolin mix of 10% cement replaced and containing gap grade with maximum size 20

MK10S10: metakaolin mix of 10% cement replaced and containing single size grade with maximum size 10

MK10S14: metakaolin mix of 10% cement replaced and containing single size grade with maximum size 14

MK15: metakaolin mortar, 15% replacement level with cement

MK20: metakaolin mortar, 20% replacement level with cement

MK5: metakaolin mortar, 5% replacement level with cement

MPa: Stress unit (N/mm2)

MSF: slump flow of mortar

OPC: Ordinary Portland cement

PCI: Precast/Prestress concrete institute

RH: Relative humidity

S/M: sand mortar ratio

S: sand

SCC: Self compacting concrete

SCHPC: self compacting high performance concrete

SCM: self compacting mortar

SFD: slump flow diameter

SI: segregation index

SP: superplasticizer

SPD: superplasticizer dosage

T_{20cm}: flow time of concrete in L-box test to until reach 20cm

T_{40cm}: flow time of concrete in L-box test to until reach 40cm

T_{500mm}: the time should be monitored when concrete reach to circle with border 500mm.

UCL:

UPFA: ultra pulverised fly ash

UPV: Ultrasonic pulse velocity

VEA: Viscosity enhanced admixture

VMA: Viscosity modifying admixture

VR: volume ratio of coarse aggregate to total concrete volume

W/B: water binder ratio

W/C: water cement ratio

W/P: water powder ratio

XRF: X-ray fluorescence spectrometry

α_a: the upper loading stress= $F_{cyl}/3$ (N/mm^3);

α_b: the basic stress (0.5 N/mm^3);

α_c: compressive strength

$ß_p$: The water powder ratio for zero flow

ΔH: height difference in J-ring test

CHAPTER 1- INTRODUCTION

1.1 General

Self compacting concrete (SCC) is an innovative and emerging development in concrete construction which is characterised as possessing high flowability, passing ability and stability but is not conditioned to be high strength or have good durability. SCC looks like “honey’’ when flowing on a horizontal level and is also characterised by a creeping movement. Therefore, the surface finishing of hardened SCC is most of the time smooth. It can flow through congested reinforced areas or areas with complex parts of structures meaning that it can fill all gaps without vibration and without the occurrence of any bleeding or segregation. It can achieve self compactibility by acquiring high deformability of paste or mortar, enough viscosity between constituent and a good passing ability through bars. It acquires high deformability of mortar by using superplasticiser and acquiring viscosity is done through the use of powder or viscosity modifying admixture (VMA), and in some cases both.

High Strength Concrete (HSC) is a type of high performance concrete with has a compressive strength of between 50 to 100 MPa (Mehta, 2006). To achieve high strength for concrete, the aggregate used does not necessarily need to be high in strength but must be compatible in terms of stiffness and strength with cement paste whilst also being highly durable. In addition, concrete can also be high in strength by using high cementitious content (cement + powder) with low water cementitious ratio (w/c), whilst the difference between the elasticity of its constituents is slight.

The combined factors for achieving high strength and self compacting concrete are the use of high cementitious content with low water powder ratio (w/p) ratios and high dosages of superplasticiser in order to achieve a good compactibility which in turn improves strength and durability.

The constituents of concretes should be selected carefully because the influence of individual constituents on self compacting concrete is more than its effect on normal concrete. Therefore, limitations should be presented regarding the properties and contents (mix proportions) of these constituents.

Powder plays a main role in producing SCC and HSC meaning that it would provide good stability for SCC as well as a good microstructure of SCC and HSC and thus would enhance the strength of the concrete. EFNARC (2002) classified powders into two types: pozzolanic materials and filler materials.

Moosberg-Bustnes et al. (2004) categorised the effect of powders on properties of concrete into three types. The first type is the physical effect which occurs as a result of pervading the fillers into voids between cement particles. Consequently, this may increase the density of cement which will lead to improving the properties of concrete in the fresh and hardened state. The second type is the surface chemical reaction effect, that is, the added particles have an indirect effect on the hydration process. This effect is induced through the particles acting as nucleation sites on the cement particles; a process which forms an integral part of the cement paste. The third type is the chemical reaction effect whereby the fillers react with some components of cement to form the cement gel. For example, pozzolan materials (fly ash, silica fume and meta-kaolin) react with $Ca\ (OH)_2$ to form C-S-H which contributes to developing the strength.

The physical properties of powders, namely particle size distribution, shape, and fineness of particles should be taken into consideration in the mix design. In cases where the powders have a surface area higher than the surface area of the cement, this will lead to an increase in water demand in order to maintain the same workability.

Concrete is mainly affected by many properties of aggregate such as type, maximum size, particle size distribution, shape and reactivity of the aggregate used (Tokyay, 1998). Aggregate affects concrete in fresh and hardened states. In SCC, aggregate has a great effect on the flowability, passing ability and stability. Coarse aggregate properties have a much less significant effect on normal strength concrete when compared to concrete with high strength. In HSC, paste has high strength due to low water-cement ratio. Therefore, aggregate properties become very important from a strength perspective (Tokyay, 1998). With this in mind, the present study focused on investigating these characteristics on fresh and hardened High Strength Self Compacting Concrete.

1.2 History

In the late 1960s, superplasticisers were developed in Japan and Germany, respectively. High strength concrete was cast in girders and beams for bridges and this was first applied in Japan. In Germany, the first application of high strength concrete took place underwater where the need was for concrete mixes with high fluidity and without segregation in order to achieve both high workability and high strength simultaneously. This can indeed be achieved through the use of superplasticisers. They were very suitable when being used in the making of casts to replace structural parts of tall buildings.

The historical development of SCC can be classified into two stages: the first development was in Japan at the end of the 1980s. In 1980, SCC was developed due to the need for producing concrete with a good durability and with less cost. One of the main reasons which led to this development was the reduction in the skilled operatives in

Japan's construction field. SCC came with the technology of self compacting for concrete when cast in the formwork. This meant that it could fill all gaps and corners under its own self weight and without using a vibrator to compact it.

In 1992, Ozawa reported the urgent need for such a type of concrete; therefore; many studies were published regarding SCC. Indeed research focusing on self placing, highly workable, highly fluidised and self consolidating concrete was carried out by Ozawa & Maekawa at the University of Tokyo. Their papers were mainly concerned with the development of the mix design of SCC with particular emphasis on the fresh properties.

Druta (2003) began the research into the development of the workability and flowability of this type of concrete. The study found that the main factors affecting the self compactibility were the properties of materials and the mix proportions.

In 1988, the sample of SCC was completed by using local materials which were readily available in Japanese markets. This sample brought into consideration the drying shrinkage, hydration heat, density of hardening and other properties. This kind of concrete was named high performance concrete and was said to involve the following stages of concrete such as self-compactable in fresh state, less initial defects in early age stage and a good durability in hardening state.

In 1989, Gagne et al. defined "high performance concrete" as a type of concrete which was characterised by high durability owing to the reduction in the water/cement ratio. Therefore, the term "high performance concrete" spread throughout the world and became known as a high durability concrete. Since this time, a number of other authors have changed this term to "self compacting high performance concrete".

In the 1990s, the use of SCC widened from Japan to other countries. One of these countries was Sweden. Sweden was the first country in Europe which started to develop SCC. In 1993, many research papers were published and were mainly concerned with the use of SCC in practical applications. The Swedish Cement and Concrete Research Institute (CBI) organised many seminars which focused on developing and using SCC for housing.

In April 1997, the Japanese Society of Civil Engineers (JSCE) compiled a subcommittee of researchers whose job it was to give special recommendations pertaining to the use of SCC in practical applications. These recommendations were published in English in 1999.

In 1998, the first worthwhile international workshop was held in Japan and concentrated on the development of SCC in a number of countries. This workshop included studies regarding development of mix design methods of SCC and studies about rheolgy.

In January 1998, the first of three bridges was constructed, outside of Japan, by using SCC. Studies conducted on these bridges showed a reduction in cost by as much as 15% in comparison with using normal concrete. Moreover, these studies revealed that the hardened properties of SCC were significantly improved when compared with those of normal strength.

1.3 Significance of High Strength Self Compacting Concrete

It is very important for any construction engineer to know about high strength self consolidating concrete as well as the advantages associated with its use.

High strength self compacting concrete is considered a crucial component in today's construction industry. The use of high strength self compacting concrete is due to the need for this type of concrete in structures with highly congested reinforcement sections and narrow corners, specifically in high buildings. The compaction process is complicated in these areas which could lead to the occurrence of weaknesses in strength of these members due to the weakness of the compaction. SCC can be placed from a height of 5 metres without the occurrence of segregation.

Normal concrete is usually compacted manually by using vibrators. There are areas where it is very difficult to reach or compact concrete in it. In some cases skilled labour must be used or the process must be supervised; an issue which brings with it many costs. The job must be carried out with high efficiency and thus bringing in untrained workers would make this process very difficult. The self compacting concrete has many advantages as shown below:

1. Faster during casting and without the need for mechanical vibration which would lead to reductions in cost.
2. Possibility to fill and form all complex architectural shapes which are not possible or difficult to form with normal concrete.
3. It has a uniform finish of surfaces.
4. Can encapsulate the reinforcement bars and has a good bond with them.
5. Improved pumpability.
6. Less construction periods and labour leading to cost reductions.
7. Reduction of noise and increased safety on work sites due to vibrator elimination.

1.4 Objectives of The Study

Due to the increased need for concrete in constructions in Malaysia in the coming years, it is expected that more studies will be conducted regarding concrete and the low cost ways in which it can be developed. Due to the limited number of studies focusing on HSSCC, the use of HSSCC in Malaysia is also limited. SCC is very sensitive in the selection of materials and also requires higher content of powder and chemical admixtures (superplastisiser and viscosity modifying) compared to normal concrete, thus resulting in increased cost with regards to this concrete. However, in large structures, the high cost of SCC materials is substituted by saving in the construction time and labour costs as well as the other advantages which have been previously shown. Therefore, it is necessary to conduct many studies in order to overcome the problems of making this kind of concrete. This study focused on these problems by using local powder with low costs whilst at the same time these powders partially replaced with cement to reduce the cost and reduce the pollution which is produced from the process of cement making. Moreover, this study used these materials to reduce the need for chemical admixtures by using a number of the powders to increase the flowability and using others to provide viscosity of SCC mixes. In addition, this study used simple design methods with less trial mixes. Other problems such as aggregate characteristics which affect SCC properties were also studied to avoid any unexpected complications in future works.

The main aims of the present study were:

1. To investigate the effect of fine aggregate properties on the workability of mortar.

2. To study the possibility of using local materials (pozzolan and inert filler materials), as suitable replacements for cement and sand respectively. This is in the hopes of achieving self compactibility and good mechanical properties for HSSCC.
3. To investigate the saturation dosage of superplasticiser of every replacement level of powders with cement or sand.
4. To estimate the differences in the response of crystalline and amorphous of kaolin and its influence on workability and strength of high strength self compacting mortar (HSSCM).
5. To find a correlation between flowability of mortar and concrete.
6. To evaluate the effect of the volume, maximum size, grading and shape of coarse aggregate on the fresh concrete properties (the filling ability, passing ability and segregation resistance) and hardened concrete properties of HSSCC.
7. To find correlationships for the various fresh tests and hardened tests of HSSCC.

1.5 Scope of the study

This study is divided into three parts which are as follows: paste, mortar and concrete. In the paste part, the water: cement ratio and the required dosage of superplasticiser are specified. In the mortar part, two pozzolans namely fly ash and metakaolin and filler (limestone and kaolin) act as partial replacements for cement and sand respectively. Four replacement levels are used for every type of pozzolan and filler which are 5, 10, 15, and 20%. In this stage, the saturation dosage of superplasticiser for every mortar mix is determined. The optimum replacement level of every type is determined depending on workability and strength. In concrete part, the effect of the coarse aggregate characteristics namely volume, maximum size, grade and shape on the behavior of HSSCC is studied.

1.6 Thesis Layout

In this study, an experimental work and statistical analysis has been carried out to achieve the above aims.

The work presented in this thesis is spread across five chapters. Chapter 1 includes a general introduction and a detailed description of the historical development of HSSCC.

Chapter 2 includes a summarised review of relevant literature regarding workability, specification; requirement, mix design methods, the constituents of HSSCC and their influence on fresh and hardened properties.

Chapter 3 presents the experimental work while in Chapter 4, test results are analysed and compared, and are accompanied by a extensive discussion. This chapter is divided into three parts which are as follows: paste, mortar, and concrete.

Finally Chapter 5 provides the conclusions derived from this study and recommendations for further investigations.

Chapter 2- Literature Review

2.1 Introduction

This chapter deals with previous studies regarding the production of HSSCC and their components as well as the different mix design methods of HSSCC. The chapter will also analyze past studies relating to the fresh and hardened properties of HSSCC.

2.2 Definitions

High strength concrete (HSC) is a type of high performance concrete. High performance concrete has high strength and is extremely durable whilst high strength concrete has high strength but does not have the same level of durability as high performance concrete (CIP33, 2001). ACI 363 1992 defined high strength concrete as having a compressive strength of more than 40 MPa, while FIB/CEB (1990) defined high strength concrete as having a compressive strength of more than 60 MPa at 28 days of demolding. De Larrard and Bostvironnois (1991) defined HSC as having compressive strength between 50 to 80 MPa at 28 days. They also stated that it contains a chemical admixture as water reduction.

Self Compacting Concrete (SCC) is a high performance concrete with low yield value which allows it to move material (low shear stress) without segregation (PCI report 2002). Goodier (2003) defined SCC as a concrete with high flowability which retains stability (without segregation). Similarly, Ashok (2004) defined SCC as highly flowable and stable; properties which enable it to fill all gaps and crevices when it is cast without consolidation and segregation. Schutter et al. (2008) defined SCC as a highly fluid and cohesive concrete in fresh state.

2.3 High strength and self compactibility of concrete

It is easy to produce SCC with high compressive strength because there are combined factors for producing HSC and SCC. HSC has a cement matrix with low capillary pores which could be a result of reducing w/b ratio and using HRWR as well as using powder materials to increase the amount of products of hydration (Nagataki and Sakai, 1994). At the same time, SCC needs low w/b ratio and high powders to achieve good stability and high dosage of high range water reducer (HRWR) to achieve high flowability. Another factor is improving the interface transition zone between aggregate matrix which is also done by using low water content and using pozzolanic materials (Mehta and Aitcen, 1990). Cementitious content should be high within the range of 415-650 kg/m^3 and w/b ratio should be between 0.23-0.35 (CIP33, 2001). By trial mixes are done to produce HSC by Mehta and Aitcin, the study showed that shrinkage and creep increase with the increased paste volume potentially; therefore, the paste to aggregate ratio is fixed 35:65 by volume.

There are two main factors which are crucial for achieving HSC. Firstly, the densification of matrix and transition zone between paste or mortar and aggregate must be high. High densification of matrix and transition zone can be achieved by reducing w/b ratio and the use of pozzolanic materials to consume more Ca $(OH)_2$ and produce more C-S-H as well as achieving high packing for matrix and thus producing high strength concrete. Secondly, the utilisation of various kinds of coarse aggregate such as limestone aggregate improves the transition zone due to the chemical action of calcium carbonates. Moreover, the use of crushed aggregate with small maximum size provides a large surface area and thus would lead to an increase in the transition zone. The poor workability due to low w/b ratio can be improved by using superplasticizer (Aïtcin, 1998, De Larrard et al, 1997, Neville, 2005).

Okamura and Ouchi (2003) reported that the self compactibility of concrete can be achieved by balancing the flowability and viscosity of paste or mortar so that the increase of flowability is necessary to reduce the friction between aggregate particles. At the same time the presence of proper viscosity is also necessary in order to avoid any segregation between the constituents of mixes particularly as the aggregate passes through the steel bars. The flowability can be increased by using HRWR and increasing viscosity either with a viscosity modifier, a mineral admixture, or both. Moreover, reducing the coarse aggregate volume in the mix can reduce the collision probability of aggregate particles and thus increase the flowability, passing ability and stability of the mix. Researchers have found that high content of coarse aggregate would consume the required energy for flowability.

Liu (2009) summarised the procedures necessary for achieving SCC in the following chart as shown in Figure 2.1.

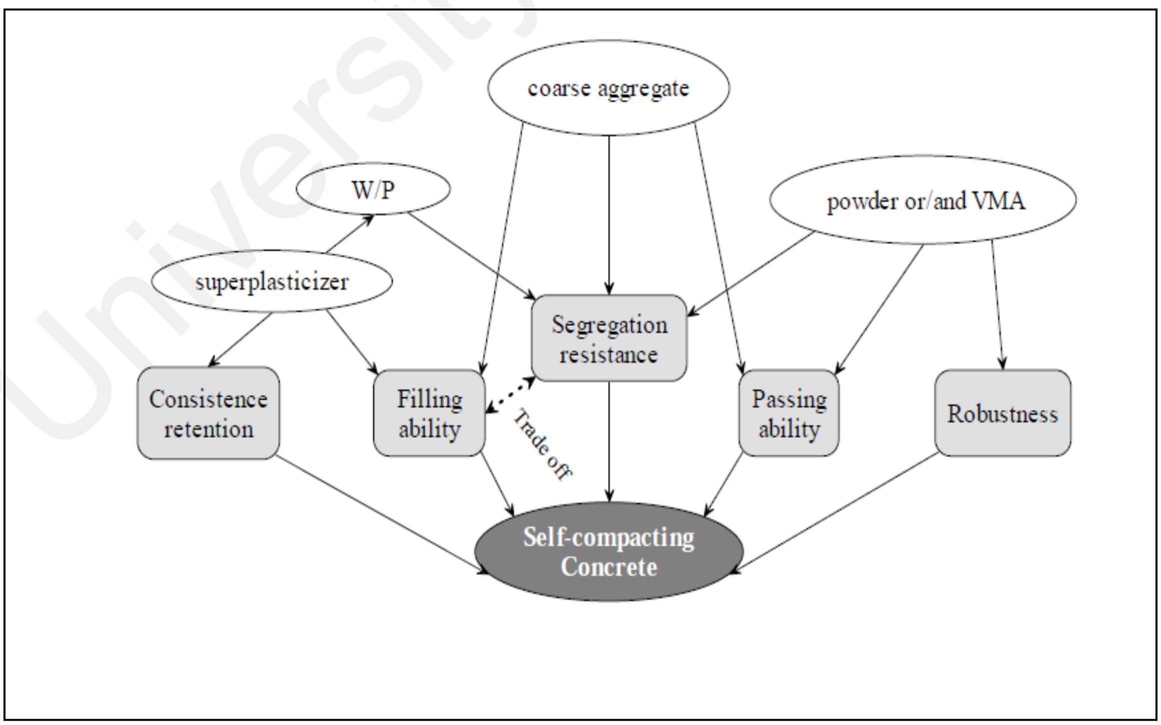


Figure 2.1: Flow chart for achieving self compactibilty of concrete (Liu, 2009)

2.4 Materials

2.4.1 Cement

Cement is considered to be the major material in producing HSSCC. Indeed, it forms a significant portion of binder content and has a great effect on workability and strength. Some of the chemical compounds of cement as well as physical properties of cement affect the workability and strength of HSSCC. For example, EFNARC (2002) determined the percentage of C_3A in cement so that it does not exceed 10%. A high percentage of C_3A would lead to poor workability retention of mixes due to the early rapid hydration of C_3A and C_4AF (EFNARC 2002, Mehta and Montriro 2006 and Collepardi, 1998). C_3A and C_4AF content affect the efficiency of dispersion of HRWR because HRWR is initially adsorbed by C_3A and C_4AF (Nawa et al., 1998). Generally, all cement types which are in accordance with EN 197-1 can be utilised (EFNARC, 2005). The particle size distribution of cement has a great affect on the rheology of paste; therefore, the cement used should improve the fluidity of the paste. The cement utilised should also be free from the problem of false setting which affects the mixing procedure after adding water (Safiuddin, 2008). The cement used should not generate high hydration heat. This will make it possible to avoid the volume changes which occur in concrete (Struble & Hawkins, 1994, cited by Safiuddin, 2008).

2.4.2 Minerals Admixtures

2.4.2.1 Introduction

Mineral admixtures are powders which are used in concrete to improve the characteristics of concrete. These admixtures are used by using cement or sand as replacements.

Moosberg-Bustnes et al. (2004) categorised the effect of powders on properties of concrete into three types. The first type is the physical effect, which occurs as a result of pervading the fillers into voids between cement particles. Consequently, this may increase the density of cement which will lead to improving the properties of concrete in the fresh and hardened state. The second type is the surface chemical reaction effect, that is, the added particles have an indirect effect on the hydration process. This effect is induced through the particles acting as nucleation sites on the cement particles and forming an integral part of the cement paste. The third type is the chemical reaction effect where the fillers react with some components of cement to form the cement gel. For example, pozzolanic materials (fly ash, silica fume and meta-kaolin) react with Ca $(OH)_2$ to form C-S-H which contributes to developing the strength.

The physical properties of powders, namely particle size distribution, shape, and fineness of particles should be taken into consideration in the mix design. In cases where the powders have surface area less than the surface area of cement, this will lead to the increase in water demand for maintaining the same workability. Burak et al. (2006) mentioned that the optimal dosage of cementitious or inert powders is based on their physical and physico-chemical characteristics. They also stated that these characteristics have a significant effect on the fresh paste properties. These characteristics assume the shape of particles, surface porosity, and fineness fraction content and particle size distribution. It is occasionally difficult to recognise them among the effects of these factors due to the complicated effect of the combination of these factors.

2.4.2.2 Fly Ash

Fly ash is a pozzolanic material which is defined as silica or silica and an alumina material. With this said, these materials react with the product of cement hydration (calcium hydroxide) and would produce compounds with cementitious properties. ASTM (316) (1993) classified the fly ash types into two kinds namely Class F and Class C. Class F is produced from burning the bitumenous coals. Class C is produced from burning the sub bitumenous coals. Class C fly ash has pozzolanic properties in addition to cementitious characteristics, while Class F has pozzolanic properties and rarely cementitious characteristics. There are many studies regarding the use of fly ash as a supplementary cementitious material. Li (1998) mentioned that the sizes of fly ash with spherical shape vary between under 1 to 100 µm. The particle size distribution and surface characteristics of fly ash affect the pozzolanic activity as well as the water demand of mixes containing fly ash. Helmuth (1986) ventured that the reason for the water reduction effect of fly ash on the mixes is that the small fly ash particles play a role in dispersing cement particles whereby it is adsorbed into the cement particles causing dispersion of these particles with behaviour similar to the behaviour of water reducing admixtures leading to increased workability.

Fly ash was used successfully in SCC (Domone, 2006) and was found to improve both the workability and strength of later ages. Sonebi (2004) concluded that the presence of fly ash as a replacement for cement in SCC reduced the plastic viscosity as well as the yield stress. In contrast, Park et al. (2005) concluded that the use of fly ash in paste led to a slight reduction in yield stress whilst at the same time increasing the plastic viscosity. Moreover, Shadle (2002) reported that fly ash decreased water bleeding and improved stability. Xie et al. (2002) concluded that ultra pulverised fly ash (UPFA) affected the workability of SCC similar to the effect of viscosity agent as well as increasing flowability without affecting viscosity (cited by Koehler & Fowler, 2007). Fly ash particles have a spherical shape which leads to a reduction in the water demand and increases the plasticity of paste or concrete. Consequently, the concrete mixes have a lower bleeding and are easier to consolidate which can lead to high strength and durability. Jiang & Malhotra (2000) concluded that the water reduction of fly ash concrete was 20% with the replacement level around 55% with cement weight. They also stated that the use of fly ash led to a reduction in superplasticizer dosages. Conversely, the water demand was reduced by 3% once 10% of fly ash was replaced with cement (Building Research Establishment 1997).

Ferraris et al. (2001) explained that particles with spherical shape have fewer surface to volume ratios. This will provide good fluidity of mixes due to the minimising of inter-particles' friction which means that these particles can roll more easily over one another. The spherical shape would achieve high packing density thus meaning that the void ratio will be less and consequently would improve the strength. Park, et al. (2005) concluded that unburned carbon in some types of fly ash may adsorb superplasticizer and thus would reduce the workability.

The effect of fly ash on the behaviour of concrete depends on the replacement with cement, that is, whether by mass or by volume. If the specific gravity of fly ash is small and if the replacement was by weight this means that the volume of fly ash paste would be more than the neat cement paste. Shadle & Somerville (2002) concluded that the use of Class F fly ash reduces the strength development of mixes at early ages with level more than the use of Class C. However, in contrast they found that it improves the durability better than Class C. The strength development can be compensated through the addition of an accelerator. Moreover, Class F also accelerates the setting time more than Class C. Fly ash plays a significant role in producing high strength concrete (Blick et al, 1974, Schmidt 1975, Ukita et al., 1992 and Klieger & Lamond, 1994, cited by Li, 1998).

2.4.2.3 Metakaolin

Metakaolin forms from kaolin after heating treatment, whereas kaolin is stable under natural environmental temperatures. Kaolin is a fine powder with a white coloration and is clay used to produce paper. Kaolinite is the mineralogical phase for $Al_2Si_2O_5(OH)_4$, the primary component of kaolin (40-70%). Other minerals except kaolin are quartz, rutile, and muscovite-like micas (Moulin et al., 2001). Kaolin has plate like particles and its size is fine (below 10 µm). Kaolin is a phyllosilicate, consisting of alternate stratums of silica and alumina in tetrahedral and octahedral coordination respectively (Figures 2.2 and 2.3) (Kingery et al., 1976).

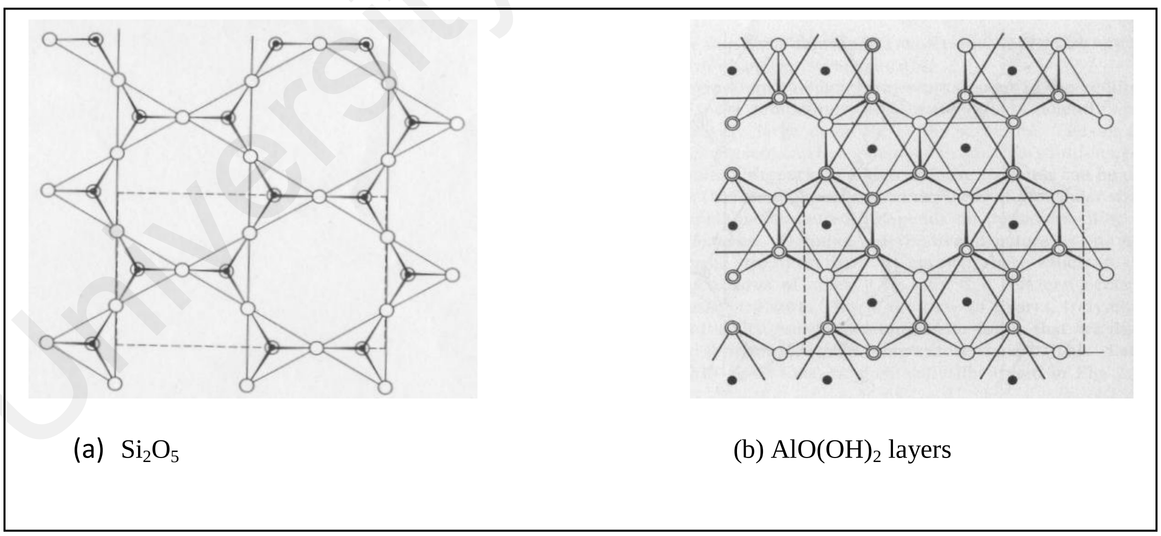


Figure 2.2: Atomic arrangements of (a) Si_2O_5 and (b) $AlO(OH)_2$ layers – (Justice, 2005)

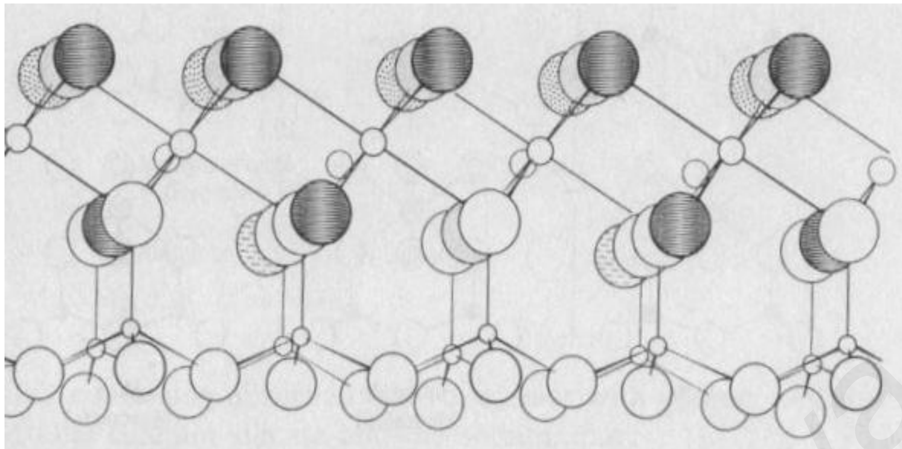


Figure 2.3: Perspective drawing of kaolinite with Si-O tetrahedrons on the bottom half and Al-O, OH octahedrons on the top half of the layer (Brinkley, 1958) cited by (Justice, 2005)

Metakaolin (MK) is produced by burning kaolin at high temperatures. It contains 40-45% Al_2O_3 and 50-55% SiO_2 (Poon et al., 2001); both of which are major components and react with calcium hydroxide to produce C-S-H. However, the percentages of other oxides are slight. MK particles are finer that cement particles. The heating required for breaking down the structure and changing it is between 650-900 °C. After burning kaolin at this temperature, kaolin loses 15% of its weight. This heat action destroys the structure of kaolin such that the silica and alumina stratum become wrinkled and lose their long range array. This dehydroxylation and disarray would result in MK. MK is considered a high reactive transition stage. MK is a shapeless pozzolan, with some dormant hydraulic properties which would make it a pozzolanic material (Bensted and Bames, 2002). Researchers found that heating to 700 °C is considered the optimal degree for burning so that the temperature below this degree results in a non reactive powder which can contain residual kaolinite. They also found that any temperature above 850 °C led to the occurrence of recrystalisation and thus a decline in the reactivity of kaolin which is then converted into an inert powder (Bensted and Bames, 2002 and Ambroise et al., 1986). The calcinations process consists of many stages. The increase of temperature of burning to 200 ˚ C would lead to removal of the absorbed water from kaolin particles. The hydrate water and interlayer water will be removed by increasing the heating until the crystalline structure transfers to disorder structure (Fook, 2004). Kostuch et al. (1993) indicated that the presence of high purity MK leads to an increase in compressive strength and a reduction in the level of $Ca\ (OH)_2$ in concrete. They also showed that all replacement levels of MK with cement (5, 10 until 20%) reduced the CH in cement paste and essentially at 20% level CH was removed completely in concrete at 28 days. Ding & Li 2002 concluded that the use of MK as a replacement for cement improved the workability with level higher than the use of silica fume in concrete.

Kostuch et al. (1993) mentioned in his study the effect of the replacement levels of MK with cement on the consumption of $Ca\ (OH)_2$. They stated that the increase of replacement level of MK led to a reduction in $Ca\ (OH)_2$ with time. The 20% replacement level of MK with cement led to the complete removal of $Ca\ (OH)_2$ in concrete at 28 days. In 1995 however, Oriol and Pera (1995) concluded that the replacement level between 30 and 40% of MK with cement led to the complete removal of $Ca\ (OH)_2$ in MK paste at fixed W/B ratio by using lime saturated water in curing the sample for 28 days.

Bai et al. (1999) reported in their study that the presence of MK in concrete negatively affected the workability of concrete mixes. Khatib and Clay (2003) mentioned in their study that the utilisation of MK in concrete increased superplasticizer dosage to obtain a specific workability and that this dosage increased with an increase of the replacement level of MK with cement. Qian and Li (2001) also concluded that the increase of MK content in concrete mixes led to a gradual loss in slump of mixes at the same dosage of superlasticizer. However, with an increase of superplasticizer dosage, the effect of increasing MK in concrete mixes on slump became minor. Wild et al. (1996) concluded that the presence of MK in concrete mixes affected the strength of MK concretes from three sides, the first side is MK acts as a filler, whereby it accelerates the hydration of OPC due to the pozzolanic activity of MK with Ca $(OH)_2$. The major impact of pozzolanic activity lasts for between 7 and 14 days. The second side is the optimum replacement level of MK with cement was 20% to provide maximum enhancement in strength over a long time. This is in contrast with the use of silica fume which continuously improves the strength up to at least 28% of replacement with cement. While, the third side is the effect of MK on the increase of strength of concrete mixes does not continue after 14 days regardless of the replacement level. This could be due to an elementary change after 14 days in the nature of the reaction between MK and Ca $(OH)_2$. While, Siddique and Klaus (2008) showed in the study that MK is an effective pozzolan, enhances the mechanical properties at all ages of all paste, mortar and concrete, decreases the water penetration of concrete through capillary action. It causes considerable changes in chemical composition of pores phase of hydrated paste and thus it would improve the pore structure of all of the cement, mortar, and concrete. This improvement in the pore structure significantly decreases the permeability of pores, thus increasing the resistance of movement of water and other structurally harmful ions which lead to deterioration of the matrix. Finally, it enhances the resistance of paste, mortar and concrete to sulfate attack. Alkali-silica reaction and this resistance increase with an increase in the replacement level of MK especially the replacement levels of 10 and 15%. Guneyisi et al. (2008) found that the utilisation of ultrafine MK with cement led to the production of high strength concrete mixes with low shrinkage.

2.4.2.4 Limestone powder

Limestone is normally regarded as an idle additive in hydrating cement pastes. However, there is a consensus that limestone powder is not pozzolanic, and many other studies have proven that its reactivity can be significant. Ramachandran (1996) proved that the addition of limestone filler to concrete accelerates the hydration process of Portland cement particles (especially the C_3S) during the early stages. Its reaction with C_3A creates a calcium carboaluminate which is formed on the surface of C_3A. Ellerbrock and Spung (1990) also indicated that adding the limestone powder to cement increases the packing density of the cement particles. Bonavetti and Rahhal (2001) believed that the limestone becomes a nucleation site for calcium hydroxide.

Husson et al. (1992), Barker and Cory (1991) reported similar trends when the addition of limestone to Portland cement enhanced the hydration of C-S-H and the formation of calcium hydroxide, possibly because it offered nucleation sites for its growth. On the other hand, Ushiyama et al. (1986) speculates that the addition of small amounts of carbonates disables the early hydration of alite, while adding large amounts accelerates its hydration. More recently, work by Evrard and Chloup (1994) has pointed out the reactivity of calcite in an alkaline environment, while confirming the hypothesis of Rarmachandran and Chun-Mei (1986), in that a small reaction occurs between calcite and C-S-H. The forming of carboaluminates and limestone's reactivity has also been documented in several other experiments.

Bombled (1986) noted that when limestone filler's influence on the rheology of fresh concrete is substantial, it is usually in the sense of better plasticity, a higher cohesion, and, in some instances, a quasi-acceleration of the setting. Neto and Campitelli (1990) performed a two-point test to characterize the rheology of limestone filler cements. They noticed that there was a reduction in the yield stress as the limestone increased, as well as enhancement of the plastic viscosity past certain fineness in its relation to limestone. Brookbanks (1989) presented the results in a comprehensive study on the effect of limestone additions to cement in the range of 5% to 28% on the properties of fresh concrete. The study found that the setting times were slightly reduced as the limestone addition increased. From the materials used, there was no discernable difference in the water demand at 5% replacement, while there was a slight reduction was observed at higher filler levels. The limestone fillers significantly reduced the water bleeding but evidentially had no effect on the air entraining properties. Brookbanks (1989) conducted an intensive study on the effect of limestone additions to cement between the ranges of 5% to 28% on the properties of fresh concrete. The study found that the setting times lightly decreased as the limestone addition went up. For the materials used, there was no noticeable difference in the water demand at 5% replacement, while another slight reduction was noted at higher filler levels. The limestone fillers greatly reduced the water bleeding and seemed to have no effect on the air entraining properties. Libre et.al (2010) observed the results of using limestone powder and fly ash as mineral admixtures, as well as a high range water reducer and viscosity modifying agent as chemical admixtures on the fluidity and viscosity of self-compacting mortar. They believed the experiment proved that the use of limestone powder and fly ash also improves fluidity, at least in some cases. Yahia et al. (2005) found that, for a given w/c and superplasticizer, the increase of limestone out of a given range leads to a rise in viscosity. Beixing et al. (2009) believed that replacing 10% manufactured sand content with limestone fine would enhance the abrasion resistance and strength of high strength concrete and led to a 10% decrease in the abrasion resistance. Sahmaran et al. (2006) found that using fly ash and limestone powder as replacement for cement improved the workability of self-compacting mortar. Limestone powder can assist the maintaining of stability in a high workability mix; however it will not add significantly to the compressive strength development of SCC (Domone and Chai, 1997, cited by Newman and choo 2003). Felekoğlu et al. (2006) found that the results gathered from compressive strength tests indicated that both limestone fillers were better than fly ash, in terms of early strength gain. However, after 28 days, mixes that included fly ash gave higher strength values than the control mixtures because of the pozzolanic effect that fly ash has. Petersson (2001) also found that limestone filler that is higher will yield an early, higher compressive strength. Lower filler content indicates a lower blocking ratio. With lower filler contents, higher admixture contents may be used to increase the blocking ratio. Al-jabri (2005) found that SCC could be accomplished through the addition of very fine mineral admixtures and a suitable superplasticizer. These two materials give the right amount of balance between the viscosity and yield of the mix. This work also found that mixes contain more fineness or more quantity of the mineral admixtures when their superplasticizer dosage is increased from 75% to 275% when compared to the reference mix. The study concluded that the effect of mineral admixtures on the fresh properties of SCC is not the same and will vary in accordance to the type, number of admixtures, dosage, fineness, as well as the particles-distribution system. As the fineness, dosage or the numbers of the mineral admixtures goes up; the water demand also is higher due to the increase in the surface area and thus the flowability decrease, and the viscosity increase. In some experiments, it was documented that the use of fine mineral admixtures can decrease the water demand or boosts the slump.

2.4.3 Chemical Admixtures

2.4.3.1 Introduction

Chemical admixtures are representative of ingredients that can be added to concrete throughout the early stages of the mixing process in order to boost some basic properties of both fresh and hardened concrete (air entrainment, stability or workability for the former, strength and shrinkage for the latter). Chemical admixtures such as retarders, superplasticizer (SP), viscosity-modifying admixtures and other high range water reducers have proven to be useful. The efficiency of chemical admixtures is indicated by comparing the varying of strengths of trial mixes. Compatibility between supplementary cementing materials and cement and water reducers should also be studied during these trial tests. As a result, it may be possible to discover the workability, bleeding, and the amount of water reduction and setting times for specified admixture dosage rates, as well as the times of addition (Druta, 2003). Kuroiwa et al. (1983) found that the chemical admixtures were included to accelerate the viscosity and deformability of the concrete. The most common chemical admixtures used to produce SCC are usually superplasticizer and viscosity modifying admixtures.

2.4.3.2 High Range Water reducer (HRWR)

High Range Water Reducer (HRWR) or Superplasticizer (SP) is a fundamental component of HSSCC when attempting to indicate the necessary workability. The four categories of HRWR, as indicated by Neville (2005), Mindess et al. (2003) are usually sulfonated melamine-formaldehyde condensates, sulfonated naphthalene-formaldehyde condensates, modified lignosulfonates, and carboxylated acrylic ester copolymers or polycarboxylates. As seen in Figures 2.4 and 2.5, high-range water reducers prevent the formation of cement-water agglomeration in concrete mixture and disperse the cement particles in the aqueous phase Therefore, the water demand of concrete mixture has decreased significantly. HRWRs can deploy a water-reducing action through two different mechanisms, known as electrical and steric repulsions (Safiuddin, 2008).

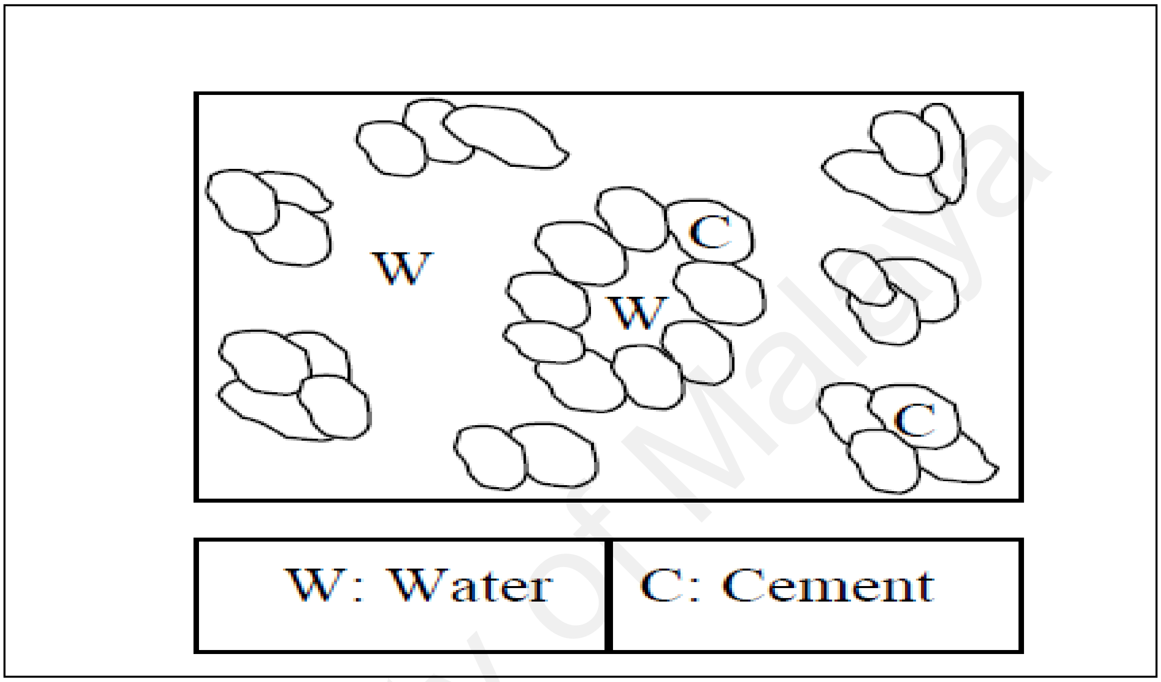


Figure 2.4: Cement-water agglomeration in absence of HRWR (Safiuddin, 2008)

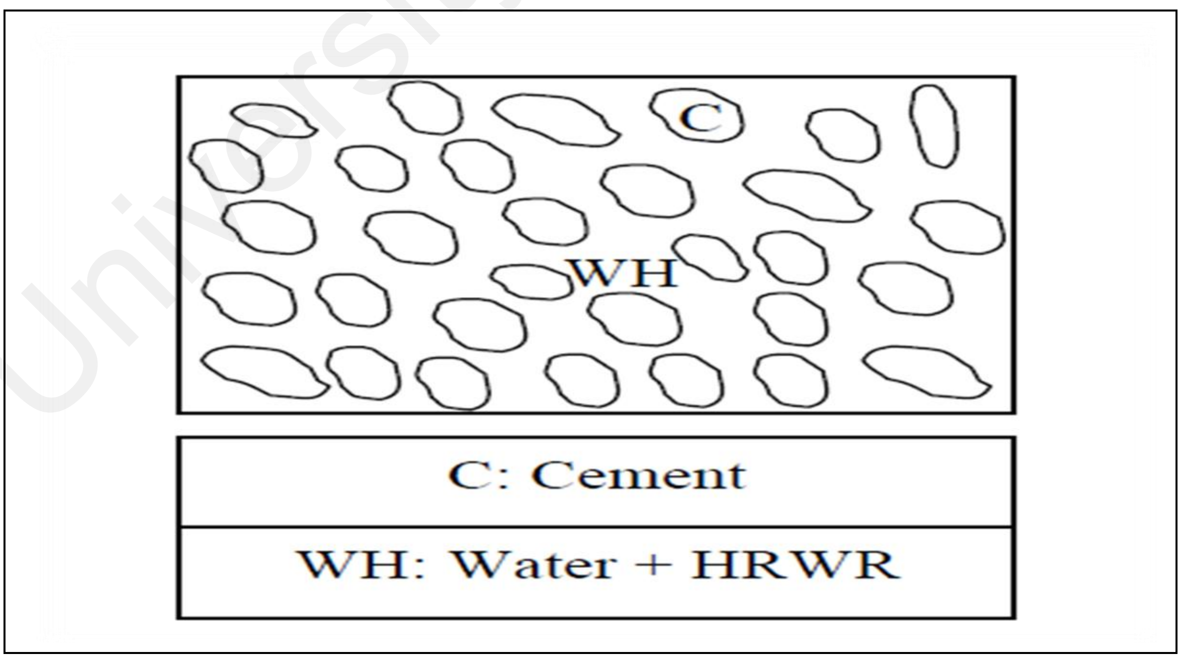


Figure 2.5: Dispersion of cement particles in presence of HRWR (Safiuddin, 2008)

First generation HRWRs deploy the water reducing action by equidirectional charging of the cement grains and reducing the surface tension of the water. The addition of HRWR to concrete causes the cement particles to disperse in the aqueous phase of fresh concrete, preventing the cement-water agglomerates from forming (Dodson, 1990). The adsorption of the HRWR molecules on the cement grains drives this dispersion mechanism. The adsorbed molecules cause a repulsive force between cement particles (electrical repulsion) because of their negative electrical charges. As a result, this prevents flocculation and the cement particles are dispersed homogeneously throughout the fresh concrete (Erdogdu, 2000). This result causes a substantial decrease in the mixing water quantity, as the water molecules, which had been incased in cement grain flakes, are now free. Second-generation HRWRs (such as polycarboxylates) work through causing steric hindrance effects (also known as the steric repulsion mechanism) to stop the agglomeration of cement-water (Bigley and Greenwood, 2003 cited by Safiuddin, 2008). It has unique graft chains (side chains) of polyethylene oxide that can reach onto the surface of cement particles (Chandra and Björnström, 2002 cited by Safiuddin, 2008). These graft chains move through water and lead to an even dispersion of the cement grains in the mixture. The main chains of polycarboxylates HRWR build around the cement grains and stop them from contacting each other. Therefore, the cement grains cannot form cement-water agglomeration, and the water needed for a given flowing ability is substantially lowered (Safiuddin, 2008).

HRWRAs based on polycarboxylates are different to sulfonate-based HRWRAs in their mode of action and basic structure. Sulfonate-based HRWRAs consist of anionic polymers that can be adsorbed into cement particles to leave a negative charge, which results in electrostatic repulsion. In contrast, polycarboxylate-based HRWRAs, are made up of flexible, comb-like polymers with grafted polyethylene oxide side chains and a polycarboxylic spine. This spine, which includes ionic carboxylic or sulfonic groups, can be absorbed onto a cement particle, while the nonionic side chains extend outward. The side chains can physically separate cement particles, which is known as a steric hindrance. Polycarboxylate-based HRWRAs can function through electrostatic repulsion as well as steric hindrance (Bury and Christensen, 2002; Yoshioka et al., 2002; Cyr and Mouret, 2003; Li et al. 2005 cited by Koehler and Fowler, 2007). The reduced significance of electrostatic repulsion is shown in less negative or near-zero zeta-potential measurements for cement pastes with polycarboxylate-based HRWRAs in comparison to cement pastes with sulfonate-based HRWRAs (Blask and Honert, 2003; Li et al., 2005; Collepardi, 1998; Sakai, et al. 2003 cited by Koehler and Fowler, 2007). In fact, zeta-potential measurements are usually not sufficient enough to justify dispersion of cement particles by polycarboxylate-based HRWRAs when we consider the DLVO (Derjaguin and Landau, Verwey and Overbeek) theory for electrostatic repulsion (Sakai, et al. 2003 cited by Koehler & Fowler, 2007). Polycarboxylate-based HRWRAs, in comparison to sulfonate-based HRWRAs, generally produce rheological characteristics that are more favorable for the production of SCC (Koehler and Fowler, 2007). Whiting (1979) found that high range water reducers could possibly decrease the net water content of mixtures from 10% to 20% when utilized in manufacturer's recommended doses. It was also found out that one- and three-day compressive strengths may be significantly raised through utilize of high range water reducers. Compressive strengths higher than 70 MPa were gathered after 28 days of curing. The drying shrinkage was lowered in the trial to decrease the net content of water in the concrete mixtures. Ouchi et al. (1996) showed that the superplasticizer affects the relationship between the viscosity and flowability of the paste to achieve self-compactability. As a result, they found that the ratio of flow speed to flow area in mini V-funnel of cement paste mixtures with a fixed amount of high range water reducer was almost constant, regardless of the water-cement ratio. An even higher amount of superplasticizer caused a lower ratio of flow speed to flow area. This ratio was thought to be an indicator for the influence of superplasticizer flowing ability and viscosity of cement paste from the standpoint of achieving self-compatibility. It was also found out that the relationship between superplasticizer amount and its effect were different, depending on the type of chemical admixture or cement.

Khayat (1998) believed that the introduction of a high range water reducer results are insignificant in yield value, plastic and apparent viscosities, and particularly in grouts containing a low dosage of viscosity modifying admixture. This study also shows that high range water reduction is often used to reduce yield value and viscosity of cement paste, therefore preventing the necessary increase in the water/cement ratio to maintain a given consistency. This study also concluded that the additional dosage of high range water reducers can disperse cement grains, which in turn liberates some of the trapped water in the grains and increases the amount of free water in the system. Ozkul and Dogan (1999) found that the use of chemical admixture, which differed slightly from conventional ones, lead to an increase in water reduction, beside to the high workability retention for longer periods of time. This research also showed that concrete containing superplasticizer can be easily pumped from a height of 13 meter, and the filling capacity was more than 85%. The pumping pressure was the same as pumping pressure of normal concrete and no bleeding or segregation was noted. The slump spread diameters were between 500 mm and 740 mm for mixes with w/c ratios between 0.3 and 0.45, and the compressive strength ranged between 53 MPa and 68 MPa at 28 days of curing age. Tviksta (2000) noted this when superplasticizer was introduced in SCC to obtain the fluidity. Nevertheless, a high dosage near the saturation amount can boost the segregation in concrete. Okamura and Ouchi (2003) showed that the requirements for superplasticizer in SCC are an effect of high dispersion for low water/powder ratios, a low amount of sensitivity to temperature changes, maintaining the dispersing effect for around two hours after the mixing process.

HRWRs can improve the flowing ability of SCHPC through liquefying and dispersing actions. They can reduce the plastic viscosity and yield stress of concrete through liquefying actions (Hu et al., 1996, Yen et al., 1999), and therefore provide a good flowing ability in SCHPC. Additionally, the HRWRs deflocculates the cement particles and free the trapped water through dispersion (Aïtcin, 1994), and therefore boosting the flowing ability of SCHPC. In this dispersing action, the inter-particle friction decreases the flow resistance, and the flowing ability of concrete is improved as a result. Al-Jabri's research (2005) found that when only a superplasticizer is used, concrete tends to segregate due to the loss in yield stress within the particles, coupled with the fact that materials with different gravities are based within the mixture. One of the main characteristics of SCC is segregation evasion, also referred to as "stability" of SCC.

Chemical admixtures, such as superplasticizers (high–range water reducer) can boost concrete strength through reduction of the mixing water requirement for a constant slump, and through dispersion of cement particles, either with or without a change in mixing water content, allowing for more efficient hydration. The main thing to consider when using superplasticizers in concrete is the high fines requirement for cohesiveness of the mix and rapid slump loss. However, neither of these is harmful in the production of HSC. HSC mixes usually have more than sufficient fines because of the high cement content. The use of retarders, along with high doses and redoses of superplasticizers at the plant or at the job site, can increase strength and restore the slump to its initial amount. Even a superplasticized mix, which can appear stiff and difficult to consolidate, is very responsive when vibration is applied (Hover, 1998, Safiudin, 2008; Peterman and Carrasquillo, 1986)

2.4.3.3 Viscosity Modifying Admixture

Viscosity modifying admixtures (VMA) are water-soluble polymers that boost the cohesion and viscosity of concrete's main components. This enhancement of the liquid-phase viscosity is essential in flowable systems in reducing the rate of separation of material constituents and improving the homogeneity and performance of a hardened product. Viscosity modifying admixtures are most commonly used along with high range water reducing agents to achieve a high, yet cohesive fluid for the main components of concrete that can flow readily into place with minimal separation of the various constituents of different densities, as well as reduce water dilution and enhance the degree of suspension of various solids. However, not all types of viscosity modifying admixture have showed the preferred result (Khayat, 1998).

Berke et al. (2002) believes that SCC should be produced without a VMA if possible, but that a VMA is necessary in certain situations, such as where aggregate moisture content cannot be controlled or in mixtures with low powder content or poorly graded aggregates. The VMAs used for SCC are commonly water-soluble polymers; however, other materials, such as precipitated silica, can also be used (Rols et al., 1999; Khayat and Ghezal, 2003; Collepardi, 2003 cited by Koehler and Fowler, 2007). Water-soluble polymers for use as VMAs in concrete are loosely classified as natural, synthetic or semi-synthetic. Examples of these are provided in Table 2.1. Common VMAs for concrete include cellulose derivatives (commonly a nonionic cellulose ether with various substitutes within it) and welan gum (a high-molecular anionic weight, natural polysaccharide fermented under controlled conditions (Khayat, 1998; Lachemi et al., 2004a; Lachemi et al., 2004b cited by Koehler and Fowler 2007).

Table 2.1: Examples of water-soluble polymers used as VMA (Khayat, 1998)

Natural	Semi-Synthetic	Synthetic
• starches • guar gum □ locust bean gum • alginates • agar • gum arabic • welan gum • xanthan gum • rhamsan gum • gellan gum • plant protein	• decomposed starch and its derivatives • cellulose-ether derivatives ○ hydroxypropyl methyl • cellulose (HPMC) ○ hydroxyethyl cellulose (HEC) ○ carboxy methyl cellulose (CMC) • electrolytes ○ sodium alginate • propyleneglycol alginate	• polymers based on ethylene ○ polyethylene oxide ○ polyacrylamide ○ polyacrylate • polymers based on vinyl ○ polyvinyl alcohol

Takada (1999) looked into the influence of welan gum as a viscosity modifying admixture on the w/c material ratio. This study found out that the viscosity modifier raised the w/c ratio because it characteristically makes the mix viscous. Welan gum increases the viscosity of the net water in the fresh concrete due to the ability of its polymers' properties to link each other in water. Dehn et al. (2000) investigated the interaction between viscosity-modifying agents and the superplasticizer from side and the bonding between the reinforced bars and self-compacting concrete from another side. It was concluded that the polymer in the viscosity modifier (welan gum) and the polymer in the superplasticizer bind each other, and this reaction can result in a rise in resistance to segregation and some larger doses of superplasticizer for some particular deformabilities. Also, they found that, depending on chemical admixtures dosages and mix design, the bond behavior in SCC was superior to the bond behavior in conventional concrete. Kayat and Guizane (1997) documented that the use of welan gum could affect the watery phase of the cement paste, where the water-soluble polymer chains could absorb some of the free water in the paste structure, therefore improving the cement paste viscosity. Because of this, less free water is obtainable for bleeding. The increase viscosity of the cement paste can also enhance the capability of the cement paste to hold solid particles, a process that cripples the sedimentation. Mixes containing a viscosity modifier manifested a shear thinning behaviour, but the apparent viscosity reduced with the raise in shear rate.

The mechanism to enhance viscosity depends on the type of viscosity enhancement admixture (VEA). The VEAs under Class A work through the mechanisms of absorption, intertwining and association (Khayat, 1998). In absorption, the long-chain polymer molecules absorb and correct a part of the mixing water (Okamura and Ozawa, 1995), and expand by adhering to the periphery of water molecules. Because of this, the freedom in movement of the water molecules is prevented and the viscosity of the mixing water and that of the concrete mixture is enhanced. In association, the molecules of adjacent polymer chains gain more attractive forces that also prevent water movement and construct a gel, boosting the viscosity of concrete. Additionally, the polymer chains can entangle and intertwine, resulting in an enhancement of the viscosity in the concrete mixture. However, such intertwining depends on the deformation or shear rate of concrete and the concentration of VEA.

2.4.4 Aggregate

SCC mixtures normally have a mixture of a smaller maximum aggregate size, lower total aggregate content, and a larger amount of fine aggregate in relation to coarse aggregate. Although SCC can be constructed with a wide range of aggregate sources, the optimization of aggregate characteristics can result in improved flow properties and reduced demand for chemical admixtures, water and cementitious materials. In choosing an aggregate source for SCC, key characteristics can include, but are not limited to, angularity, shape and texture; grading (including the maximum aggregate size); and microfine characteristics. In predicting flow properties, these characteristics may be considered based on rheology or empirical-based models (Koehler and Fowler, 2007).

2.4.4.1 Coarse Aggregate

All types of coarse aggregate are suited for producing SCC. The aggregates should be chosen by considering the performance required for fresh and hardened concrete. Aggregates gathered on the 4.75-mm (No.4) sieve are deemed "coarse aggregates" (ASTM C125, 2004). These are granular materials, like gravel or crue, and are normally combined with fine aggregate and cementing material or binder to make concrete. Coarse aggregates are also a key component of SCHPC, as with any concrete. Coarse aggregates substantially affect the performance of SCHPC by influencing the segregation resistance flowing ability, and strength of concrete (Noguchi et al., 1999, Okamura and Ozawa, 1995, Xie et al., 2002). The optimum coarse aggregate content should meet the proper properties when it is used in SCC. Where, the smaller the ultimate size of aggregate, the larger the proportion of coarse aggregate. A more substantial aggregate content can be used in comparison to crushed aggregate in SCC. The coarse aggregate content in SCC is vital in ensuring that the mix has proper mechanical properties sufficient flow characteristics (Khayat et al., 1999b, cited by Newman and choo, 2003). A high coarse aggregate content can lead to dissipation in segregation resistance and flow blockages (Okamura et al., 1998, cited by Newman and choo, 2003). Yurugi et al. (2000) noted that the coarse aggregate content has a profound effect on concrete's filling capacity. The coarse aggregate volume of SCC was believed to be about 50% of the total solid volume (Okamura et al., 1998). Geiker et al. (2002) indicated that the viscosity and relative yield stress of concrete significantly increases with increased coarse aggregate (CA) volume fraction. Hu and Wang (2011) found that, regardless of the mortar composition, concrete yield stress and viscosity are usually boosted with an increased coarse aggregate (CA) content. Amparano et al. (2000) found that a boost in aggregate content decreases the workability of a concrete mix and therefore dissipates the strength. Suwanvitaya et al. (2006) found that the total aggregate content had varying effects on the compressive strength and splitting tensile strength, decreasing the strength at a lower aggregate content and raising at the higher ranges. Aitcin and Mehta (1990), Baalbaki et al. (1992) showed that the elastic modulus of concrete was affected by the volume fraction and elastic properties of aggregates. Cho et al. (2000) believed that the elastic modulus of cement-based composite increases with the rising volume fraction of aggregates.

Aggregates may have a significant influence on the properties of hardened concrete. The deformation properties of concrete are particularly affected by aggregates due to a combination of the effects, including volume concentration water demand, aggregate stiffness and paste/aggregate interaction (Alexander, 1996). Aitcin (1998) believed that it's important to note that, although an excess of coarse aggregate could decrease drying shrinkage, it will boost the amount of microcracks within the paste. Tviksta (2000) also stated that it is possible to use naturally rounded, crushed, or semi-crushed aggregates to make SCC. Combined aggregate grading is the most important aspect of making an SCC mix, but a good gradation of solids helps to stop segregation (Avery, 2004). A well-graded aggregate source can be key in successfully producing an economical SCC. SCC can be produced with poorly graded aggregate, but this then requires additional viscosity to prevent segregation (Neuwald, 2004). Gap- grading omits one or more intermediate-size fractions. However, these intermediate-size particles are required when suspending larger-size particles and resisting aggregate segregation and suspending larger-size particles in SCC (Neuwald, 2004). EFNARC (2002) noted that gap grading is usually better than those continuously grading, which can lead to greater internal friction and reduced flow. Hu and Wang (2011) came to the conclusion that graded aggregate can considerably reduce yield stress and viscosity of concrete. Zia et al. (2005) found that HSC can be obtained by using coarse aggregate with a maximum size range of 20-25mm, although some of researchers found sizes 10 to 12mm were preferable. However, the type of coarse aggregate also has a significant role in producing HSC (cited by Majuar, 2003). The use of limestone as coarse aggregate in HSC gives a 20 percent higher compressive strength than the use of granite aggregate due to the indirect chemical interaction of limestone with some of the chemical compounds of cement (Rached, 2009). Basalt aggregate concretes also have a higher compressive strength than limestone aggregate concretes. Smaller aggregate sizes are also regarded as being able to produce higher concrete strengths due to less severe concentrations of stress around the particles, which are caused by differences between the elastic modulus of the aggregate and the paste (Rached, 2009). Alexander and Prosk (2003) concluded that the shape and size of coarse particles have a substantial influence on the required mortar and paste content. Naturally rounded river gravel requires a lot less paste or mortar than limestone. Granite needs the highest mortar volume. Crushed aggregate can improve the strength because of the interlocking of the angular particles, however reduced flow, whilst rounded aggregate, improves the flow due to the low amount of internal friction. The maximum size of aggregate is an important factor in achieving SCC. Though a smaller maximum size aggregate mixture would require more paste and a larger maximum size of aggregate would provide better mechanical properties, smaller sizes can be very suitable for the casting of heavily reinforced structures. Petersson (1997) stated that the suitable maximum size of aggregate for producing SCC is between 10mm-20mm. The passing ability goes down when the maximum size is raised. This needs to decrease the coarse aggregate content. The choice of a higher maximum size is therefore possible, but is only justified with low reinforcement content. Larger proportions of top-size aggregate may also lead to aggregate blocking in heavily reinforced structure elements. For coarse aggregate, the use of a large maximum aggregate size lowers fresh concrete water demand; however, the hardened properties can be affected negatively due to the increased interfacial transition zone thickness and the fact that larger particles tend to contain more internal defects that would usually be removed during crushing (Aitcin, 1998). In general, it is desirable to use the largest particle practical to maximize the ratio of volume to surface area (Hudson, 2003a). Hudson (2003b) quotes data indicating that mixtures with the same specific area, but different grads have similar compressive strength and water requirements.

In addition, the gradation affects the volume content of the coarse aggregates that will be used in SCHPC. The volume content of coarse aggregates can be enhanced when they are well-graded (Okamura and Ozawa, 1995). It also indicates that a lesser amount of mortar is required for well-graded coarse aggregates to achieve the passing ability and target filling ability in SCHPC. Okamura and Ouchi (2003) discovered that the flow speed of concrete through a funnel with an outlet width of 55mm was largely affected by the grading of coarse aggregate. Rahim (2005) found that the flowability of SCC was reduced with a rise in the maximum size of coarse aggregate and volume ratio. He also found that segregation tendency for mixes with a larger size of 20 mm aggregate is higher than a small size of 10 mm aggregate. The coarse aggregate is usually stronger than the paste; its strength has never been a substantial factor for normal strength concrete. However, the coarse aggregate strength does become more of a factor in the case of higher-strength concrete or lightweight aggregate SCC (Ahamad, 1994). Mehta and Montoero (2006) conducted tests that indicated that the compressive strength and elastic modulus were affected by the mineralogical characteristics of the aggregates. Crushed aggregates from limestone and fine-grained diabase produced the best results. Concretes made from crushed granite that contained soft minerals and smooth river gravel were shown as being weaker in strength. Aulia and Deutschmann (1999) concluded that the strength of the interface zone can be raised by using the smaller nominal maximum aggregate size. The use of strong aggregates is required to avoid concrete failure at a low level of stress. Bager et al. (2001) conducted research which proved that increasing the aspect ratio of particles increased yield stress and plastic viscosity.

Shape and texture of coarse aggregate is evidentially not as important as shape and texture of fine aggregate, but affect the behaviour of fresh and hardened concrete. Shape and texture affects the demand for sand. Angular, elongated, flaky, and rough particles have high voids and require more sand to fill them in order to provide workable concrete, which, in turn, increases the demand for water (Legg, 1998). These poorly shaped aggregates also produce mixtures that are prone to segregation. Elongated and flaky particles tend to produce harsh mixtures and affect finishability (Legg, 1998). According to Shilstone (1990), flaky and elongated particles, principally those of intermediate sizes (minus 3/8 (9.5mm), plus No 8 (2.36 mm)), damage the mobility of mixtures and contribute to harshness. An excess amount of badly shaped particles could have a negative effect on the strength of concrete because of the increase of water demand. Additionally, flat particles can be oriented in such a way that they could disrupt the durability and strength of concrete (Galloway, 1994; Popovics, 1979) Flat particles near the surface of concrete pavements inhibit bleed water from entering mortar above particles, therefore leads to the deterioration of the surface (Kosmatra, 1994).

2.4.4.2 Fine Aggregate

The grading, particles shape and fineness modulus's of fine aggregate are all significant factors in the making of SCC. The grading of fine aggregate in the mortar should be such that both stability and workability are maintained simultaneously. Using crushed sand is important for blocking behaviour. With crushed sand, the slump flow (yield stress) will be less and the T_{500mm} (time required for the concrete flow to reach a circular shape with 50cm diameter) (viscosity) higher with the same amount of admixture. The use of crushed sand also requires a larger dosage of superplasticizer to achieve the workability required. SCC usually requires high fine content and a viscosity modifying admixture. Another important factor that affects SCC rheological behavior is the sand to total aggregate ratio (sand/aggregate ratio). Ashok (2004) found that flowability boosted the sand/aggregate ratio and elastic modulus and is not significantly affected by sand/aggregate ratio when the total aggregate volume is kept constant. Al-Jabri (2005) concluded that the coarse sand may be unsuitable for SCC due to the amount of bleeding it promotes. Ravindrarajah et al. (2003) found that a surplus in the fine aggregate content increased the cohesion of the self-compacting concrete. The amounts of fine and coarse aggregate must be balanced, e.g., excess sand requires more cementitious materials, makes pumping harder, produces sticky mixtures, causes crazing and finishing problems (Shilstone, 1990) and increases bleeding and permeability (Mindess and Young,1981). On the other hand, insufficient sand can produce "bony" mixtures and various other finishing problems (Shilstone, 1990; Mindess and Young, 1981). Bager, et al. (2001) concluded that increasing the sand fineness boosted the yield stress and plastic viscosity of self-consolidating mortars.

2.4.5 Water

Water should always be tested for suitability in accordance with ASTM C94 (2004). Ma and Dogan, (2002) found that reduction of the water-cement ratio dissipates porosity and causes refinement of capillary pores in the matrix. However, decreasing the water-cement ratio may have a negative influence on the flowing ability of fresh concrete. Special attention should be paid to the extremely high flowing ability required in self-compacting concrete. SCC is more sensitive to the water content in the mix than more traditional vibrated concrete because of its need to measure the moisture content in the aggregates and to take into account the water content in the admixtures before adding the remaining water. Emborg (2000) showed that aggregate moisture absorption and changes in slump flow could be related to one another. This study also indicated a strong influence on slump flow when the amount of water was changed by ± 5 kg/m^3 (equaling a change in moisture content by 0.7 % of sand), see Figure 2.6 through adding a viscosity agent, these variations were then limited.

2.5 Mix Design Methods

When designing the mix, it is quite useful to put in consideration the relative proportions of the key constituents by volume rather than mass. Typical ranges of quantities and proportions that can help achieve self-compactability are given later. However, further modifications might be required to meet strength and other requirements. Water/powder ratio by volume should be with range 0.80 to 1.10, normally water content does not exceed 200 litre/m^3. Total cementitous content should be within 160 to 240 litres (400-600 kg) per m^3. Normally coarse aggregate content is between 28 to 35 % by volume of the mix. Finally, the fine aggregate content balances the volume of the other components. This previous method is depended on a method improved by Okamura and Ozawa (1995).

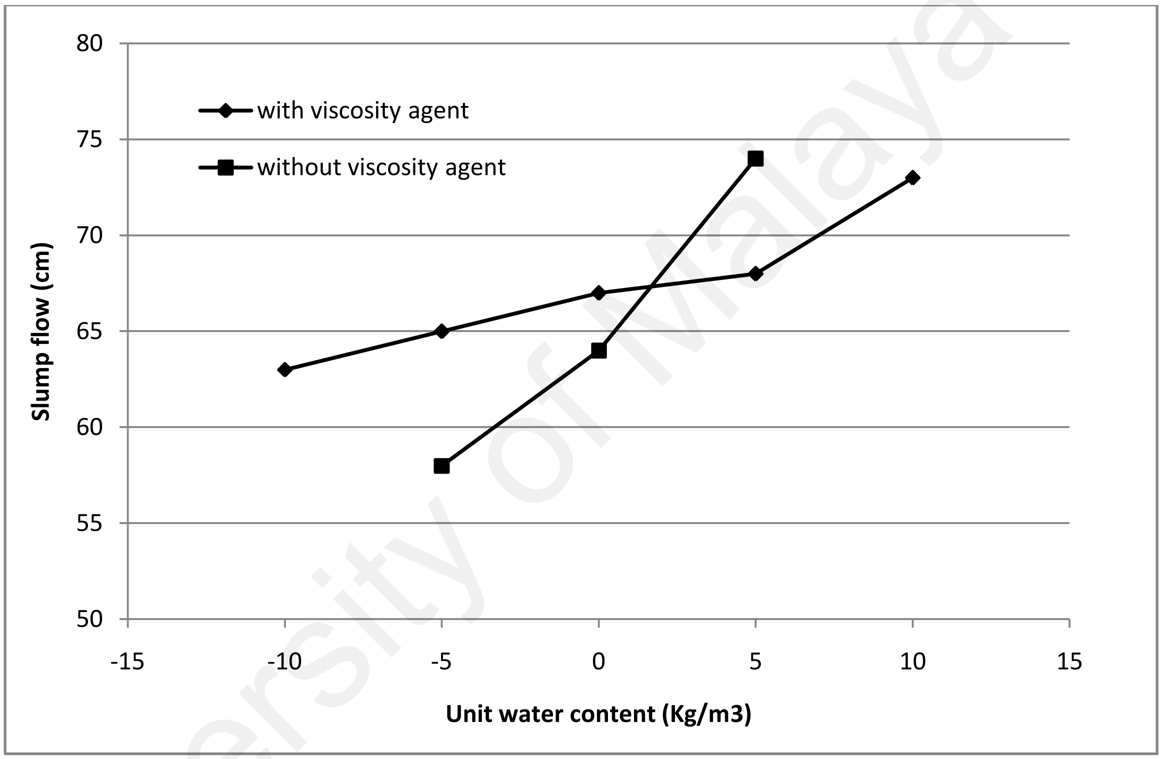


Figure 2.6: Slump flow with and without viscosity agent when water content is varied (water cement ratio 0.53) (Emborg, 2000).

This developed method put limitations of SCC constituents. Where, air content can be set at 2 %, or higher values were recommended when freeze and thaw resistant concrete is being designed. Coarse aggregate volume should be maintained at 50 to 60 percent. When the volume of coarse aggregate in concrete goes beyond a certain limit, the opportunity for collision between coarse aggregate particles rapidly increases and there is a higher risk of blockage when the concrete passes through the spaces between steel bars. The optimum volume content of fine aggregate in the mortar can vary between 40 and 50 percent, depending on the paste properties. Design of the paste was divided into stages depending on constituents of paste. The water: powder ratio for zero flow (ßp) is specified in the paste with the selected proportion of cement and other additions. Flow cone test with water/powder ratios by volume of 1.1, 1.2, 1.3 and 1.4 can be conducted with the selected powder composition as shown in Figure 2.7 for common results. The point of intersection with the y - axis is designated by the (ßp) value (EFNARC, 2002).

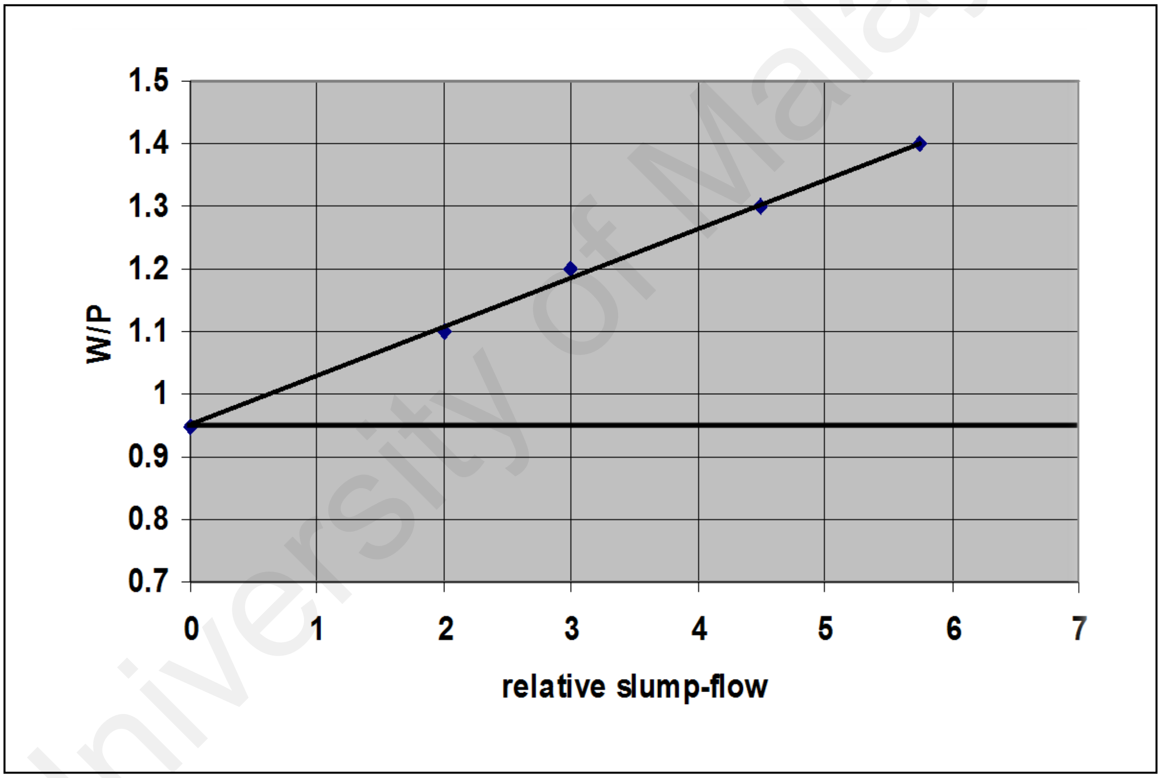


Figure 2.7: Determination of water to powder ratio ßp (EFNARC, 2002)

This $ß_p$ value is generally now used for quality control of water demand for new batches of fillers and cement. Determination of optimum volume of water and superplasticizer dosage in mortar is by testing flowability of mortar. Tests involving flow cones and a V-Funnel for mortar are conducted at varying water/powder ratios within 0.8 to 0.9 with ßp and superplasticizer dosages. This superplasticizer dosage is used to balance the rheology of the paste. The volumetric content of fine aggregate within the mortar stays the same as what is indicated above. Target values of self compacting mortar are a slump flow of 24 to 26 cm and a flow time of V-Funnel test is within 7 to 11 seconds. At target slump flow, where V-funnel time is lower than 7 seconds, then lower the w/p ratio. For target slump flow and V-funnel time in excess of 11 seconds, then w/p ratio should be higher.

If these criteria have not been reached, then this particular combination of materials may be substandard. A trial with a different superplasticizer is a preferable alternative but the addition of a new additive is another alternative worth testing.

Su et al. (2001) suggested a simple mix design method. The main focus of their theory is filling voids in loosely filling mineral aggregate with a paste of binder. The procedure is summed up and as mentioned in the next phrases. Firstly, the coarse and fine aggregate content should be determined. Computing the cement content was the next step and then computing the mixing water by basing on the cement and mineral admixture content. After calculating the water content, the superplasticizer dosage should be calculated. In order to balance the water in the mixture, the aggregate moisture should be taken into account. Finally, adjusting mix proportion is by trial mixes and SCC properties test.

In Japan, Okamura and Ozawa (1995) improved a mix design method by basing on the properties of the materials involved and the proportions they were mixed in. The total aggregate content is fixed whilst the water-binder ratio and superplasticizer content is altered to achieve self-consolidation in the fresh concrete. Typical design was detailed and as following in the next phrases. Firstly, the content of coarse aggregate should be fixed at 50% of the solid volume of the total concrete volume. The determining of the fine aggregate content was the next step and it should be fixed at 40% of the mortar volume. The determining of w/c ratio was the third step of the mix design and it is assumed inside

0.9- 1.0% by volume (the binders' properties). The final w/c ratio and superplasticizer dosage should be worked out to ensure self-consolidation.

In the mixes design of conventional concrete, the w/c ratio should be first fixed from the standpoint of gathering the required strength with self-compacting concrete. While, the w/c ratio must be settled on, taking into consideration self-compactability as it is so responsive to this ratio. (Okamura et al., 1998).

The paste or mortar in self-compacting concrete needs both high deformability and high viscosity. This can be obtained by using a superplasticizer, which leads to high deformability with a low water/ powder ratio. The characteristics of powder and superplasticizer significantly influences mortar properties, therefore the proper water/ powder ratio and superplasticizer dosage needs to be determined through trial mixing. When the mix proportion has been determined; self-compactability has to be tested through a system of U-flow, slump-flow and funnel tests. Methods for judging whether the w/c ratio or superplasticizer dosages are smaller or larger than the optimum value by using the results of fresh tests, and ways for evaluating the optimum values are required. The relationship between the mix proportion and the property of the mortar in SCC then has to be investigated and formulated. This formula can be used to establish a rational technique for adjusting the superplasticizer dosage and w/p ratio to reach the required level of viscosity and deformability.

Petersson, et al. (1996) has constructed a model for the mix design of SCC. The model was divided in stages and as shown later. The first model was determining the minimum paste volume for total aggregate by determining the void volume for combination of fine and coarse aggregates by using a modified ASTM 29 method. The minimum paste volume should be filling the gaps between aggregate particles whereas encapsulating all particles surface. The second model was the determining of total aggregate volume of non-blocking concrete mixture which was based on maximum aggregate size and the grading. The third model was determining the optimum mix proportions of mortar which can be obtained by altering the w/p ratio, high range water reducer, viscosity modifying agent and sand content until the required plastic viscosity and yield shear stress are gained by using a viscometer. The fourth model was determining the optimum mix proportions of concrete which can be obtained by computing the maximum total aggregate volume of a mix without the occurrence of blocking.

El-Ariss (2002) suggested that in order to produce SCC, the following requirements were suggested. Coarse aggregate should be less than 50% of total aggregate weight as well as fine aggregate should be 50% of total aggregate weight. Paste volume should be around 38% of the mix volume and cementitious content should be between (400-500) kg/m^3. Finally, Superplasticizer should be around 1.0 to 3.0 litres/100 kg of cement.

2.6 Limitations

The performance of SCC can be an indicator of the aggregate properties. For instance, the University College London suggested the workability recommendations for self compacting concrete containing 10mm maximum aggregate size, slump flow spread between 600 to 700 mm, flow time through V-funnel box between 2 to 4 sec, and height difference in U-box is between 300 to 350 mm are recommended. Whilst, in the case of SCC made with 20mm maximum aggregate size, the value of slump flow spread can be 650 to 700mm, flow time between 4 to 10 sec, and height difference between 30 to 350 respectively (ACI Committee 211H, 2006). Typical acceptance criteria for SCC are shown in Table 2.2 according to ACI Committee 211H (2006).

Table 2.2: Typical acceptance criterion for SCC (ACI Committee 211H, 2006)

Workability characteristic	Test methods	Recommended values suggested in 1 to 6
Deformability and flow rate (filling ability) (unrestricted flow)	Slump flow	1- Authors: 620 to 720 mm. 2- EFNARC: 650 to 800 mm.(MSA 20mm). 3- JSCE: 600-700 mm. 4- PCI:≥ 660 5- RILEM TC 174 :N/A 6- Swedish concrete association:(650 - 750)
	T-50	2- (2 to 5) sec 4- (3 to 5) sec. 6- (3 to 7) sec.
Passing ability (narrow-opening passing ability confined flow restricted flow dynamic stability	V-funnel	1- <8 sec. 2- (6 to 12) sec. 4- (4 to 10) sec.
	L-box (h2/h1)	2- >0.8 4- >0.75 6- >0.8
	J-ring	2- <10mm 4- <15 mm

2.7 Fresh Properties

Ferraris (1999) notably defined workability as the properties of fresh concrete and mortar, which indicate how much they can be compacted and manipulated. Workability is dependent on water content, additives aggregate (shape and size distribution) and age (degree of hydration). The consistency loss within concrete can be affected by several factors, such as the dosage and type of superplasticizer, w/p ratio, cement fineness and type, mineral admixtures and fresh concrete temperature.

The workability of SCC is normally superior to high performance concrete's class of workability. Workability can be characterized through following properties (Ozkul and Dogan, 1999). The first characteristic is filling ability (confined flowability) which is defined as the ability of SCC to flow under its own weight (without vibration) into and all spaces completely within an intricate framework, containing obstacles and reinforcements. The second characteristic is passing ability which is defined as the ability of SCC mixes to flow through openings approaching the size of the mix coarse aggregate, i.e., the spaces between reinforcing bars, without aggregate segregation blocking or blocking. (This property is of concern only to those applications that involve placement in sections with closely spaced reinforcement or complex shapes). The third characteristic is stability (segregation resistance) which is defined as the ability of SCC to remain homogeneous during transportation, as well as during and after placement. The stability of SCC is divided into two types; static stability type which is defined as the resistance of the fresh concrete to bleeding, segregation and surface settlement after casting while the concrete is still in its plastic state. Dynamic stability type is defined as the resistance of the fresh concrete towards the segregation of concrete constituents during placement into the formwork.

The quick and sudden loss in workability can be reduced by many ways, the replacement of the filler with cement, using retarder agent, changing of supperplaticizer type, and using cement with slow hydration rate. Because of the high fluidity of SCC, the risk of blocking and segregation is substantially increased. Preventing segregation is therefore a top priority of the control regime. Segregation can be reduced through three ways; using of sufficient amount of powder with fineness less than 0.125mm, or using viscosity modifying admixture, or using a combination of both. Increasing the volume of the paste or the mortar also leads to increase in the stability of concrete mixes. Finally, the important factor to increase the segregation resistance is by reducing the water content.

The risk of blocking occurs when a concrete flows close to obstacles. The coarse aggregate could be obstructed by the obstacle and begin to shear the mortar. If the design of mortar is not well, the coarse aggregate have a tendency to form arches and stop the flowing of fresh concrete as shown in Figure 2.8. Blocking can be impaired by increasing the spacing of the steel reinforcement bars, increasing the volume of the paste or the mortar, and finally reducing the maximum size of coarse aggregate (EFNARC, 2002).

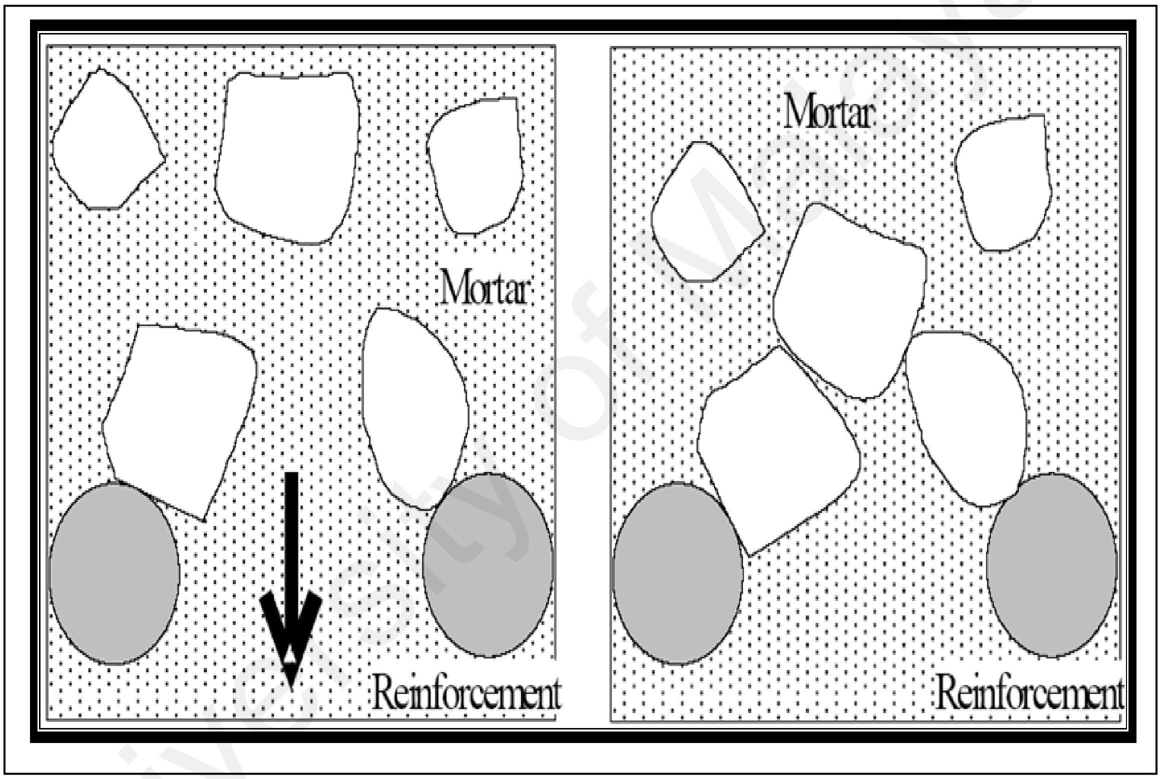


Figure 2.8: Arching phenomenons during flow in congested area (EFNARC, 2002)

2.7.1 Flowability

The filling ability mirrors the deformability of SCC, i.e. fresh concrete's ability to change its shape under its own weight (Khayat et al., 1999b; Okamura and Ozawa, 1995). Deformability involves two things: the deformation capacity is the limit to which it is allowed to deform, that is, how far concrete can flow; and deformation velocity is in reference to how long it takes for the concrete to finish flowing, that is, how fast concrete can flow. The filling ability is a balance between deformation velocity and deformation capacity. For example, concretes with large deformation capacity and very small deformation velocity tend to be quite viscous and would take a substantial amount of time to fill the framework (RILEM TC 174 SCC, 2000).

SCC needs to flow into the intended area without segregation. To achieve a high filling ability (refer to Figure 2.3), it might be appropriate to weaken inter-particle friction among solid particles (coarse aggregate, sand and powder) in concrete by using a superplasticizer and a lower coarse aggregate content (Khayat, 1999b; Sonebi and Bartos, 2002) as indicated by Figure 2.1.

The addition of water could enhance the filling ability by weakening inter-particle friction, but could also lower viscosity, leaving a danger of segregation. Too much water also has unpleasant effects on durability and strength. W/p ratios that are too small and too large can both result in poor filling ability. Unlike adding water, which lowers both the viscosity and yield stress, incorporating a superplasticizer not only weakens the inter-particle friction by dispersing cement particles but also keeps the viscosity and deformation capacity intact. It also has much less of an effect on hardened properties than water. Distribution of particle size also has an effect on filling ability. Inter-particle friction can be weakened through continuous use of graded materials, powder and aggregates (Khayat, 1999a; Sonebi et al., 2001). The filling ability, or, simply put, the free deformation degree in the absence of obstructions, is commonly estimated through its slump flow value. However, the free flow, not restrained by any boundaries, is a sole material property of the concrete. The intrinsic flow behaviour of concrete may possibly deviate when narrow passages of a framework or heavily reinforced structures need to be flown through by the SCC mix. Japanese literature contains the first example of the slump flow test (Nagataki et al., 1991) which assessed the spreading behaviour of highly flowable and underwater concrete.

In terms of physicality, the filling ability is linked directly to the yield stress of the mixture. The yield stress is the rheological parameter that influences the flow stoppage of the concrete during casting. This point of flow stoppage is gathered when the stress generated by gravity during casting (or during slump flow) equals the yield stress of the material (Roussel and Cussigh, 2008).

The EFNARC (2005) indicated that the three different slump flow classes (SF) need to be distinguished and is shown in Table 2.3. In total, a range from 550mm - 850mm in spread flow needs to be considered.

Table 2.3: The slump flow classes according to prEN 206-9 (2007) and EFNARC (2005)

Slump flow classes SF	Mean slump flow diameter
SF1	550 -650
SF2	660-750
SF3	760 - 850

Different kinds of applications are recommended; depending on the SF. SF1 class concretes are applicable for slightly reinforced or unreinforced concrete structures. This type of concrete does not qualify for long horizontal flow. SF2 class mixes may be suitable for most of the standard SCC applications, but SF3 mixes are typically produced with a small maximum size of aggregates of less than 16 mm. This concrete is used for vertical applications in complex shapes with in quite congested structures. SCCs belonging to SF3 will commonly give a better surface finish than SF2 class mixes, but also have a stronger tendency towards segregation.

Filling ability is identified as the ability of fresh SCHPC to flow into and fill the spaces within the framework in unconfined conditions and under self-weight (Bartos, 2000, EFNARC, 2002). It is associated with the formability, finishing ability and self-leveling capacity of SCHPC. The filling ability is a fundamental property of SCHPC in achieving the self-consolidation capacity. This property is crucial for concrete placement with proper casting technique (ACI 237R-07, 2007). The filling ability depends primarily on the aggregate content, w/b ratio, HRWR dosage of concrete and binder content (Okamura and Ozawa, 1995). A good filling ability can be obtained through limiting the coarse aggregate content and increasing the amount of cementing materials, while adding a proper dosage of HRWR.

2.7.2 Passing ability

Passing ability is a unique aspect of SCC. It indicates how well the mix can flow through constricted openings and narrow spaces, which ensures its particular applications in thickly reinforced structures such as bridge decks, tubing segments, abutments or tunnel linings. It depends on the risk of blocking which may result from the interaction between obstacles and constituent materials.

When SCC is used in structures with packed reinforcement, it must pass smoothly between the bars with no blocking. Blocking is a distinct result from the interaction among aggregate particles and between aggregate particles and reinforcement. When concrete is near a narrow space, the different flowing velocities of the mortar and coarse aggregate can lead to a locally increased content of coarse aggregate (Noguchi et al., 1999; Okamura and Ouchi, 2003). Some aggregates may arch or bridge at small openings, which can block the rest of the concrete, as documented in Figure 2.9 adapted from RILEM TC 174 SCC (2000).

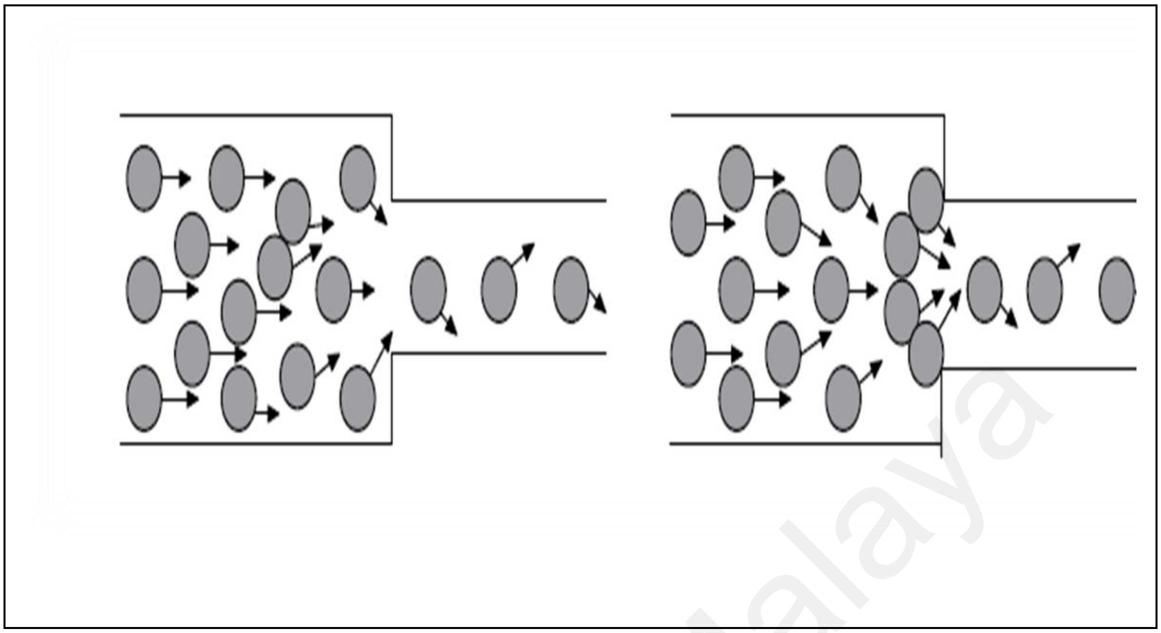


Figure 2.9: Schematic of blocking (RILEM TC 174 SCC, 2000)

Therefore, blocking is dependent on the size, content and shape of coarse aggregate (Okamura, 1997). A reduction in coarse aggregate content and reducing the size can both effectively prevent blocking. The concrete's paste volume is also an important factor in regarding blocking (Billberg et al., 2004). Another finding of Billberg et al. (2004) is that blocking depends mainly on the yield stress, whereas plastic viscosity does not affect the passing ability of SCC. However, a paste with sufficient viscosity can also prevent local increases in coarse aggregate and therefore evade blocking. By incorporating powders such as fly ash, GGBS and limestone powder, viscosity is enhanced because of better particle packing and distribution (Edamatsu et al., 1999).

As indicated by Figure 2.1, passing ability is therefore achieved through a reduction in coarse aggregate content and size, as well as the use of VMA or proper selection of powder.

Passing ability is thought of as the ability of fresh SCHPC to flow through tight spaces and openings confined by steel reinforcing bars (EFNARC, 2002 and Bartos, 2000). Where structures are densely reinforced, a quality passing ability of SCHPC enables it to be placed and consolidated through heavy reinforcing bars without the risk of aggregate blockage (ACI 237R-07, 2007). The factors influencing the filling ability also affect the passing ability of concrete. Additionally, the passing ability is dependent on the spacing and number of the reinforcing bars. A good passing ability can be obtained through increasing the filling ability of fresh concrete and limiting the segregation of coarse aggregates (Safiuddin, 2008).

2.7.3 Stability

Segregation resistance is identified as ‘stability’. Since SCC is composed of materials of varying size and specific gravities, it is highly prone to segregation. Segregation includes that between water and solid, paste and aggregate or between mortar and coarse aggregate in both flowing and stationary states (RILEM TC 174 SCC, 2000).

The resistance to segregation in SCHPC refers to its ability to remain uniform during and after placement, without any loss of stability due to mortar separation, bleeding or coarse aggregate settlement (EFNARC, 2002). The distribution of aggregates becomes non-uniform if SCHPC segregation resistance is insufficient. This could influence the durability and properties of concrete. Recent studies concluded that the water absorption and chloride penetration of SCHPC can be influenced through poor segregation resistance (Daczko, 2002 cited by Safiuddin, 2008). Good resistance to segregation can be obtained in SCHPC by a proper mixture composition. A wider range and number of cementing materials, a small nominal maximum size of aggregate, a low w/b ratio and a limited content of well-graded coarse aggregates should provide good resistance to segregation. (Bonen and Shah, 2005 cited by Safiuddin, 2008). Additionally, the segregation resistance of SCHPC can be increased by using VEA (Okamura and Ozawa, 1995).

Free water, which moves freely throughout the concrete and cannot attach to the solid particles, is the main influence on segregation (Ozawa et al., 1992). Segregation which occurs during the placing process is known as “dynamic segregation”. After placement, if coarse aggregate settles and the free water rise to cause bleeding, this is known as static segregation. Bleeding water reaches the concrete surface, or becomes trapped under obstacles such as reinforcement bars or coarse aggregate, which decreases the interfacial zone and results in impaired durability and strength.

As indicated in Figure 2.1, effective segregation resistance includes binding extra free water by a lower w/p ratio, use of VMA or a high volume of powder, providing proper viscosity to ensure homogeneous flow. A limit in the size and content of coarse aggregate is also an effective segregation inhibiting measure (Liu, 2009).

SCC typically showed minimum surface bleeding due to its high viscosity and low water content (Khayat and Assaad, 2002, and Daczko, 2002). In particular, the use of viscosity modifying admixtures and fine filler materials can enhance the ability of the paste to keep water and thus result in reduced bleeding (Khayat, 1998). However, the movement of water through SCC led to pressure gradients, causing segregation yet when surface bleeding is not there (Khayat and Assaad, 2002 and Daczko, 2002).

2.8 Hardened Properties

The behavior of hardened concrete can be characterized through analysis of its short-term and long-term properties. Short-term properties include strength in compression, bond, tension, and modulus of elasticity and the long-term properties include shrinkage, creep, behavior under fatigue, and durability characteristics such as porosity, permeability, freeze-thaw resistance and abrasion resistance (Ahamad, 1994)

Hardened properties of SCC can vary with the mix design. The reasons for possible differences between conventional concrete and the hardened properties of SCC are presented in the following text. The first reason is the use of high content of mineral admixtures (ultrafine materials) with superplasticizer. The second reason is the microstructure of SCC is better and homogeneity due to that the pores is lower and distributed more evenly resulting from low w/b ratio (Ma and Dogan, 2002).

2.8.1 Strength

Compressive strength is concrete's most important mechanical property. In general, for a given set of aggregates and cement, and under the same mixing, testing and curing conditions, the compressive strength of a concrete basically depends on w/b ratio, binder/aggregate (b/a) ratio, mixture composition and degree of consolidation. However, the w/b ratio chiefly controls the development of compressive strength in concrete. The limits of w/b ratio in achieving a targeted compressive strength in high-strength HPC are as follows (Lessard et al., 1995):

50 MPa – 75 MPa, for $0.30 \leq w/b \leq 0.40$

75 MPa – 100 MPa, for $0.25 \leq w/b \leq 0.35$

100 MPa – 125 MPa, for $0.20 \leq w/b \leq 0.30$

125 MPa and above, for $w/b \leq 0.20$

Ordinary concrete usually produces compressive strength in between 20 to 40 MPa. The compressive strength of SCHPC is much more substantial than that of ordinary concrete.

For a compressive strength varying from 50 to 125 MPa, the range of W/B ratios is also valid for SCHPC (Persson, 2001). It has also been noted that the kinetics of strength increase are notably faster in SCHPC (Persson, 2001) as compared to ordinary concrete because of the increased gel/space ratio at lower w/b ratio.

The compressive strength, generally one of the most important properties of hardened concrete is the characteristic material value for the classification of concrete in national and international codes. Because of this, whether the differences in the mixture composition and dissimilarities in the microstructure affect the short and long term load bearing behavior is of interest.

Because of the low powder/water ratios associated with SCC, the compressive strength of SCC is usually more significant than that of conventional concrete. When normal concrete is vibrated, water tends to move towards the surface of the coarser particles where porous and weak interfacial zones have developed. If SCC has been well produced and designed, it will be homogeneous, resist the segregation, mobile and will be able to be placed into the form without the need for compaction. This will allow minimal interfacial zones to develop between the coarse aggregate and the mortar phase. Therefore, the microstructure of SCC is expected to be enhanced, promoting impermeability, strength, durability and, ultimately, a longer service life of the concrete (Goodier, 2001 and Gagne, 1989). Holschemacher and Kluge (2002) found that through a simultaneous reduction in filler content and an increase in cement content could enhance the initial and maximum strength of the concrete. Tivksta, (2000), and Holschemacher and Kluss (2002) concluded that if limestone powder is used, higher compressive strengths will be noticed at the beginning of hardening process.

With conventional concrete, the proportion between cylinder and cube compressive strength (fcu, cube (150mm)/fc, cylinder (150/300mm)) is about 1.2. This relation is essentially lower for SCC; the ratio (fcu, cube (150mm)/fc, cylinder (150/300mm)) is in the range of 1.0 to 1.1. Due to this, the compressive strength's relationship to the slenderness of the specimens is significantly decreased. Dehn et al. (2000) documented the time development of the compressive strength for self-compacting concrete, as shown in Figure 2.10.

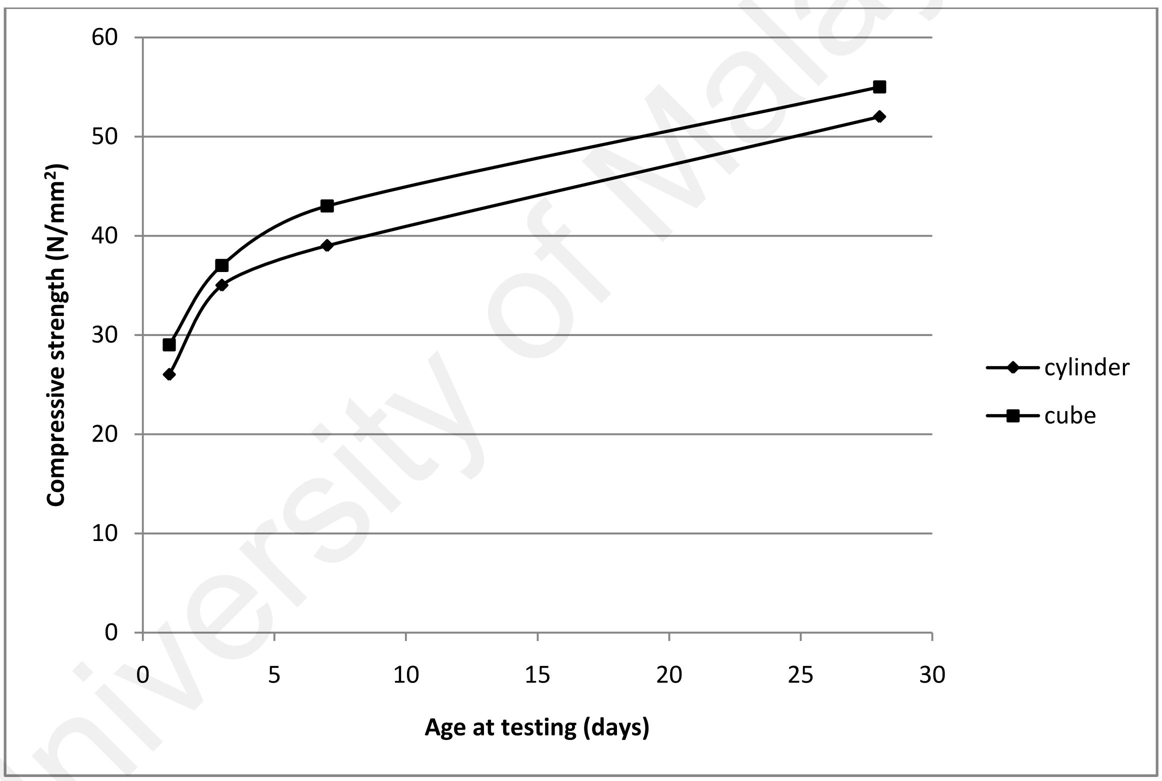


Figure 2.10: Time development of the cube and the cylinder compressive strength of SCC (Dehn et al., 2000)

The direct measurement of the tensile strength of normal concrete is not simple due to the complicated set-up that must be used. Therefore, tensile strength is usually calculated using the measurement of the modulus of rupture (ASTM C 79) and / or the splitting tensile strength (ASTM C 496) or other indirect measurements.

Holschemacher and kluss (2002) found through use of a self-compacting concrete with different types of filler such as limestone powder, fly ash, silica fume, quartzite filler and blast furnace slag, that all parameters which affect the characteristics of the microstructure of the cement matrix and of the interfacial transition zone (ITZ) are of particular importance in respect of the tensile load capacity behavior. They found the majority of results for the measured splitting tensile strength values are in the same radius of accurate regulations for normal vibrated concrete with a similar compressive strength. However, in about 30% of all measured data points, a higher splitting tensile strength was reported. The reason for this fact is given by the best microstructure, especially the smaller total porosity and more even pore size distribution within the interfacial transition zone of SCC. Ouchi et al. (2003) concluded that the 28 days splitting tensile strength of self-compacting concrete ranges from 2.4-4.8 MPa when the compressive strength is 40-80 MPa. Splitting tensile strength is about 8- 10% of the compressive strength at 28 days of age. Dehn et al. (2000) showed the time development of the splitting tensile strength for self-compacting concrete, as indicated in Figure 2.11.

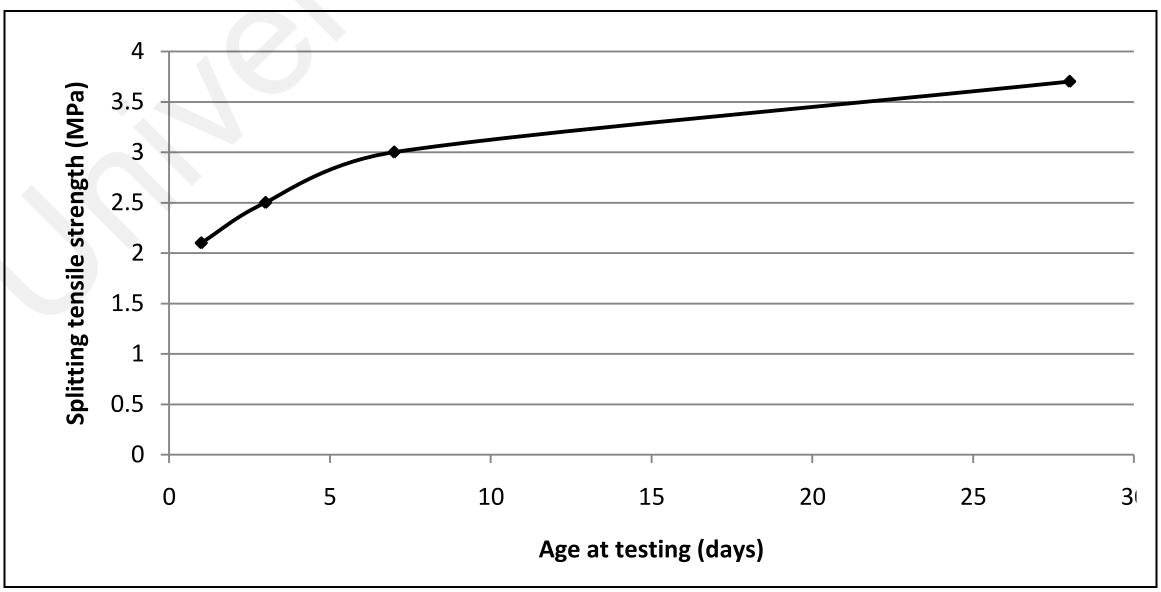


Figure 2.11: Time development of the splitting tensile strength of SCC (Dehn et al., 2000)

2.8.2 Dimension Stability

Concrete is fundamental building material whose usefulness is dictated by its exceptional durability. Therefore, this durability must be maintained. One aspect of durability under consideration is the material's volume change, or shrinkage. Excessive concrete shrinkage can lead to cracking and subsequent infiltration of harmful materials and a shorter life time.

The risk of shrinkage includes both early-age autogenous and longer term drying shrinkage may be greater for SCC due to its substantially higher paste content. SCC's highly refined pore structure could potentially boost the risk of autogenous shrinkage. The high cementitious materials content and low water-cementitious ratios can increase the susceptibility to thermal volume changes. When evaluating the susceptibility of SCC to cracking due to volume changes, the tensile strength and viscoelastic properties of concrete must also be evaluated. The higher volume changes sometimes associated with SCC may not necessarily result in the increased risk of cracking due to higher tensile strength, lower modulus of elasticity, and higher creep sometimes associated with SCC. The major factors that may affect drying shrinkage (beyond the exposure conditions and element geometry) are the aggregate characteristic and the complete content of paste and water.

As drying shrinkage is nominally the result of the loss of absorbed water from the paste, higher paste volumes and total water contents are associated with increased shrinkage (Kosmatka et al., 2002). Increasing the water-cement ratio at constant cement content or increasing the cement content at a constant water-cement ratio will increase drying shrinkage, although this increase is largely caused by the higher paste volume.

Bissonnette et al. (1999) concluded that water-cement ratio had a minor effect on shrinkage when the paste volume was held constant; however, increasing the paste volume at constant water-cement ratio caused excessive shrinkage. The fineness and composition of cement generally has a negligible effect on drying shrinkage (Mehta and Monteiro, 1993; Koskatka et al., 2000). In addition, SCMs usually have little effect on drying shrinkage. Accelerators and some water reducers can boost drying shrinkage. The use of aggregates with high stiffness and low shrinkage decreases drying shrinkage (Mehta and Monteiro, 1993; Kosmatka, et al., 2002; EFNARC, 2005). Other aggregate characteristics can indirectly influence shrinkage by controlling the amount of the water and paste needed in the mixture (Mehta and Monteiro, 1993; ACI Committee 209 R-92 2004).

The drying shrinkage of SCC may be higher than in conventionally placed concrete because of the higher paste volumes (Hammer, 2003; EFNARC, 2005). However, the drying shrinkage of SCC may be reduced because of the denser microstructure (Klug and Holschemacher (2003). The total water content of SCC mixtures may be no greater than incomparable conventionally placed concrete. As gathered from an examination of results from around the world, Klug and Holschemacher (2003) found that the drying shrinkage of SCC was typically10-50% higher than that predicted by the CEB-FIP model code. Turcry, et al. (2002) found that the drying shrinkage strains of two different SCC mixtures were similar to conventional, comparable mixtures due to the offsetting effects of increased paste volume and reduced water-powder ratio. Suksawang et al. (2005) measured increased drying shrinkage in SCC in comparison to a comparable conventional mixture. Roziere et al. (2005) found that total shrinkage in SCC (including drying and autogenous) was enhanced continuously with paste volume and that limestone filler and fly ash (to a smaller extent) decreased drying shrinkage.

Attiogbe et al. (2002) found that lowering the sand-aggregate ratio dissipated drying shrinkage of SCC. Persson's findings (2001) indicated that, at a consistent compressive strength, drying shrinkage had similarities in SCC and normal placed concrete. Bui and Montgomery (1999a) concluded that lowering the water-binder ratio and paste volume and the use of limestone filler could reduce the drying shrinkage of SCC; however, the fresh properties had to be appropriate for no segregation and good compaction. Heirman and Vandewalle (2003) found that when a range of fillers were mixed into SCC without altering the cement/water ratio or the cement content, the shrinkage was enhanced when compared to conventionally placed concrete. In comparing a conventional mixture with a SCC mixture with similar water-cement ratios (but with larger powder content in the SCC), Vieira and Bettencourt (2003) documented that the shrinkage was identical on virtually all levels.

Concrete shrinkage is a volume change of concrete resulting from environmental and structural factors. Shrinkage of concrete occurs in two stages: early and later ages. The early stage involves the first 24 hours of concrete life, whereas long-term age represents the concrete at an age of and beyond 24 hours. Shrinkage immediately starts after the water and cement come in contact during concrete mixing. These early-age volume changes are normally ignored in the design of concrete structures as it was historically believed to be much less than shrinkage resulting from drying. Also, concrete is still quite moist during its initial stages, making it tough to perform shrinkage measurements on this unsettled material. These difficulties derail determining the factors affecting plastic shrinkage. Drying shrinkage in its early stages can be avoided through proper curing techniques just after placement, to provide time for the strength of the material to build and prevent loss of moisture. The shrinkage types are shown in Figure 2.12 (Holt, 2001).

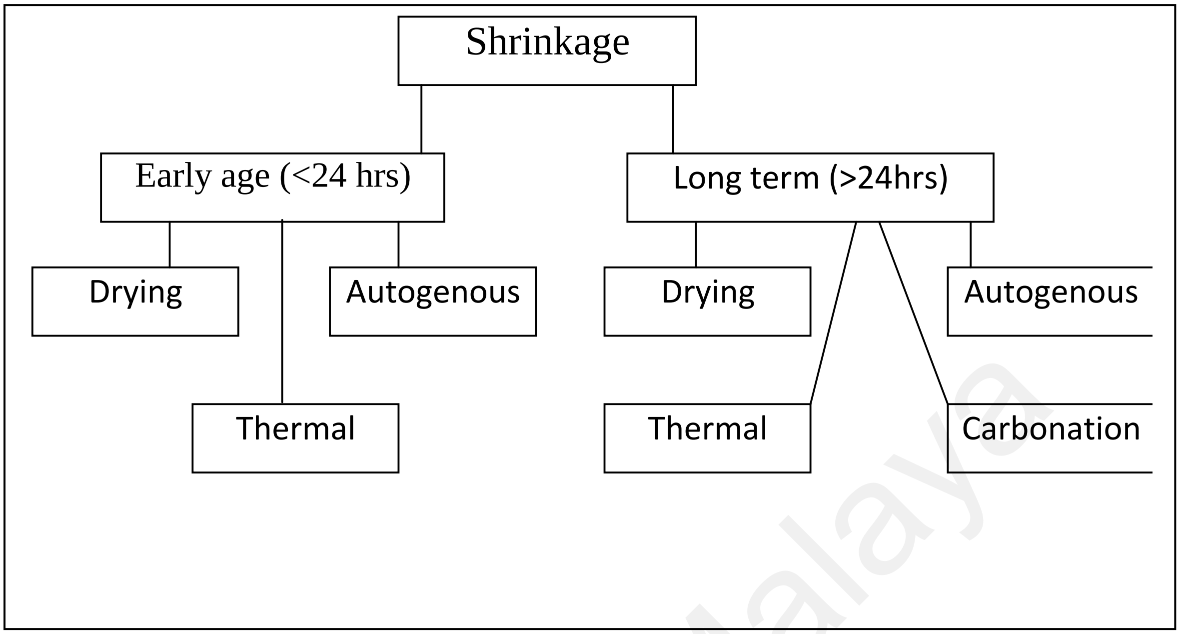


Figure 2.12: Diagram of shrinkage stages and types (Holt, 2001)

Holschemacher and Kluss (2002) reported that drying shrinkage of SCC is affected by w/c ratio and the type of sample curing with the same as normal concrete. The coarse aggregate content used can assist to decrease the shrinkage, although, a minimum paste volume has to be in place, to guarantee an optimal SCC without segregation. An intensive microstructure of the cement paste can be performed by adding fillers with high fineness compared to that of cement, where the shrinkage volume is influenced positively. The drying shrinkage of SCC is higher than that of normal concrete with about 10 to 50%. Holt and Schodet (2002) reported that SCC has lower drying shrinkage than that of high strength concrete. In fact, this is considered true in case of use of fly ash or limestone as fillers as well as the certain dosage of superplasticizer because of the high filler added in SCC leads to reduce in the drying shrinkage so that filler act as restraint factor.

Drying shrinkage is often due to the loss of water from the concrete to the atmosphere. More often than not, this loss of water is from the cement paste, but with a few types of aggregate the main loss of water is from the aggregate. Drying shrinkage is quite slow and the stresses it induces are partially balanced by tension creep relief. The aggregate cripples the shrinkage of the cement paste, and so the higher the volume of the aggregate and the higher E-value of the aggregate, the more decreased the drying shrinkage will become. A decrease in the maximum aggregate size resulting from a higher paste volume increases the drying shrinkage (EFNARC, 2005).

As concrete compressive strength is related to the water cement ratio, in SCC with a low water/ cement ratio drying shrinkage reductions can go beyond it. Tests were performed on the shrinkage of different types of SCC, and a reference concrete indicated that the deformation caused by shrinkage may be higher (Rahim, 2005).

Holschemacher and Klug (2002) indicated that a big influence on the shrinkage deformations seems to result from the aggregate combination, particularly in relation to coarse, fine aggregates, as well as fineness (Blaine) and content of ultra fines. So the shrinkage can be reduced by a higher content of coarse aggregates. However, a minimum paste volume must be present so an optimal self-compaction of SCC without segregation is ensured. Furthermore, a denser microstructure of the cement paste can be achieved by the addition of fillers with fineness larger than that of cement, whereby the shrinkage dimension is positively affected.

Holt and Schodet (2002) proved that SCC has lower shrinkage than other standard high strength concretes. This was particularly true when using limestone or fly ash as filler along with certain superplasticizers. The high amount of filler added in SCC compared to reference concrete resulted in lower shrinkage due to the restraint provided by stiffening paste before setting time.

2.8.3 Durability

Absorption is a process of filling the pores of the porous bodies by liquid by which a liquid gets into and tends to fill the open pores in a porous solid body, such as the components of concrete (ASTM C 125, 2002). The absorption is commonly more substantial in surface layer rather than the core of concrete because of strong capillary action. The rate at which a dry concrete surface absorbs a liquid can be taken as a predictor of the durability of concrete. Water is the most common liquid with which the concrete comes in contact. Therefore, water absorption is widely used to investigate the absorptivity of concrete. It can be determined based on the boost in mass of a concrete specimen due to the penetration of water into its open pores.

Water absorption is directly related to concrete's resistance to water penetration, which plays a significant role in various deterioration mechanisms and moves many deleterious agents away from the surroundings. Like other engineering properties, the water absorption of concrete is affected directly by the porosity (Hearn et al., 1994). The porosity controls the microstructure and therefore the absorption of concrete, depending on the relative quantities of the pores of various types and sizes (Hearn et al., 1997). When the porosity is decreased, the water absorption is also reduced. It was discovered that SCHPC provides water absorption within the range of 3 and 6% (Schutter et al., 2003, Vanwalleghem et al., 2003).

CHAPTER 3- EXPERIMANTAL WORK

3.1 Introduction

With a constant focus on the objectives of the research, the following experimental program consisted of carrying out necessary tests pertaining to high strength self compacting mortar (HSSCM) system and high strength self compacting concrete (HSSCC) system.

The aim of this study was to investigate the effect of coarse aggregate and mineral admixtures on the properties of SCC. Four different types of mineral admixture were used whilst three different characteristics of aggregate were studied.

Tests were conducted in order to ascertain the differences in the behaviour of HSSCM and HSSCC in both its fresh and hardened states. The mini slump cone, V- funnel box instruments and setting time test were used to evaluate the fresh properties of mortar. Compressive strength, density and absorption tests were conducted in order to assess the hardened properties of mortar. The slump flow, V-funnel, J-ring, L-box, segregation sieve and fresh density tests were conducted for concrete during its fresh state. Destructive and nondestructive tests were also conducted in order to assess the quality of concrete in its hardened state. The destructive tests used were compressive strength, flexural strength, and splitting tensile strength. The non-destructive tests consist of static modulus of elasticity, ultrasonic pulse velocity (UPV), and absorption. Furthermore, the drying shrinkage of SCC was also investigated.

3.2 Materials

It was essential that optimum proportions be selected according to the mix design methods, taking into consideration the characteristics of all materials used. Satisfactory

HSSCC was achieved by selecting suitable materials, good quality control and proportioning.

3.2.1 Cement

The cement used in this study was Tasik brand Type I ordinary Portland cement. Cement was packed in bags with weights of approximately 50 kg. It was then stored in airtight steel drum containers to protect it from exposure to moist conditions. The cement was tested and monitored according to the Malaysian Standard Specification MS 522:1989 requirement. Five tests were conducted namely X-ray Fluorescence (XRF) analysis, Laser Particles Size Distribution (PSD), Scanning Electronic Microscope (SEM), X-ray Diffraction (XRD) and specific gravity tests.

3.2.2 Fly Ash

Fly ash was collected from Kapar power plant. It is grey in colour and was packed in bags with each bag weighing approximately 50 kg. It was stored in airtight plastic bags and kept in plastic containers to protect it from exposure to moist conditions or undesired contaminants. There were five tests conducted namely X-ray Fluorescence (XRF) analysis, Laser Particles Size Distribution (PSD), Scanning Electronic Microscope (SEM), and specific gravity tests.

3.2.3 Kaolin and Metakaolin

The kaolin used in this study was supplied by Kaolin (Malaysia) Sdn Bhd. The kaolin used is called “refined kaolin” as a trade brand. The refined kaolin was packed in bags weighing approximately 25 kg each. Kaolin was stored in airtight plastic containers in order to protect it from exposure to moist conditions or undesired contaminants.

Metakaolin was prepared by burning local refine kaolin in an electrical rotary furnace. This furnace was programmed so that the temperature can increase from room temperature to 300 $^{\circ}$C in a 1/2 hour. The temperature was increased to 700 C in 3 hours and maintaining the temperature for a further 7 hours. Following this, it was left to cool overnight after which time it was ready for collection and storage in plastic containers in order to protect it from exposure to moist conditions or undesired contaminants. The electrical furnace used is shown in Figure 3.1. There were five tests conducted namely X-ray Fluorescence (XRF) analysis, Laser Particles Size Distribution (PSD), Scanning Electronic Microscope (SEM), and specific gravity tests.

Figure 3.1: The electrical furnace

3.2.4 Limestone

Limestone powder was supplied by Omya Malaysia Sdn Bhd with the name (BETOSARB) product. The limestone was packed in bags weighing approximately 25 kg each. It was stored in airtight plastic containers to protect it from exposure to moist conditions or undesired contaminants. Five tests were conducted namely X-ray Fluorescence (XRF) analysis, Laser Particles Size Distribution (PSD), Scanning Electronic Microscope (SEM), and specific gravity tests.

3.2.5 Fine aggregate

The fine aggregate used was silica sand from L&T Minerals Sdn Bhd. Each bag of silica sand weighed approximately 50 kg whilst five different sizes were used, namely 4-6 mesh, 8-16 mesh, 16-30mesh, 30-60mesh and 50-100mesh. The five different sizes were blended in order to obtain a fineness modulus of 2.5.

3.2.6 Coarse aggregate

The coarse aggregate used in this study was crushed granite which was directly collected from Kajang Rock quarry. For the purpose of the experimental work, coarse aggregate was divided into many sizes namely 19mm-14mm, 14-9.5mm and 9.5-4.75mm. The various sizes were blended in accordance with the requirements of maximum size and grading of this study and in accordance to BS 882:1992. The required tests for aggregate were conducted i.e. water absorption, specific gravity and grading according to B.S. 812: Part 812: 1990. The flakiness index was tested according to B.S 812: Section 105.1:1989.

3.2.7 Water

The water used in this study was tap water which is free from impurities that can affect the properties of both mortar and concrete samples.

3.2.8 Chemical Admixtures

3.2.8.1 Superplasticizer (SP)

The chemical admixture used in this study was a polycarboxylic based water reducer and plasticizer. Description of superplasticizer properties are listed in the following Table 3.1.

Table 3.1: Technical description of Sika viscocrete-2055

Form	Colour	Relative density	Water reduction
Off white liquid	brown	1.05 @ 24 C^{o}	30%

3.2.8.2 Viscosity Modifying Admixture (VMA)

The VMA used in this study was produced by Sika Company which is branded as Sika®.Stabiliser-4R MY**.** The description of this brand is shown in Table 3.2.

Table 3.2: Technical description of Sika®
Stabiliser-4 RMY

Form	Colour	Relative density	Dry content
Cloudy liquid	clourless	1.01 @ 24 C^{o}	3%

3.3 Mixture preparation

3.3.1 Mix design procedures

The mix design of HSSCC must satisfy the criteria of filling ability, passability and segregation resistance. HSSCC mixes must be designed to have a 28 day characteristic compressive strength $40 \leq \alpha_c \leq 100$ MPa. The mix design method used in the present study is compared to the European guidelines (EFNARC,2002) as well as a number of methods for producing HSSCC. Firstly, design of paste by determining the optimum w/c ratio and superplasticizer dosage by testing the flowability of paste. Secondly, design of mortar by determining the optimum fine aggregate proportion by testing flowability and strength of mortar. Finally, determining the optimum proportion of coarse aggregate in concrete by testing workability and strength.

3.3.2 Variables and Limitations of HSSCC design

The major variables of study which affect mix proportions of mixes are four. Firstly, types of powders (pozzolan as a replacement for cement and filler as a replacement for sand). Secondly, the replacement levels of pozzolan and fillers from 0% to 20% of cement and fine aggregate weight respectively. Thirdly, superplasticizer dosages (SPD) vary from 0.5 to 2.5% of binder weight. Finally, coarse aggregate characteristics namely (maximum size (20, 14 and 10mm), grading (continuous, gap and single), and shape (low and high flakiness ratio). While, the limitations of mix design of this study are described as following, air content does not exceed 2% of concrete volume, superplasticizer dosages do not exceed 2% of binder weight, w/p ratio is equivalent to 0.32 by weight, maximum fine aggregate size is 4.75mm, and maximum coarse aggregate size is 19mm, minimum slump flow is 600mm, minimum compressive strength is 40 MPa.

3.3.3 Design approach

The mix design passed through many stages from cement paste to concrete. The cement paste is thought of as the vehicle which transports the aggregate. Thus, achieving self compactibility of the cement paste is necessary in order to achieve the self compactibility of SCC. Therefore, there must be balance between the flowability and viscosity of paste by selecting the proper ratio or dosage of w/p and superplasticizer respectively. The paste cement design was designed according to EFNARC guidelines (EFNARC, 2002). The paste cement design included the determination of w/c ratio and optimum dosage of superplasticizer of neat cement mixes. Cement content was assumed as 500 kg/m^3.

The w/c ratio was determined for all mortar and concrete mixes. The specified w/c ratio was in accordance with the paste design method of EFNARC (EFNARC, 2002) whereby the selected w/c ratio represents the minimum water demand for cement particles to allow the cement paste to initiate flow. The required w/c ratio was 0.32 by weight or 0.995 by volume. This ratio is in the range of criteria which will be mentioned in Section 3.3.5.

The optimum superplasticizer dosage was determined based on the results of the mini slump flow cone of cement paste mixes and was 1.1% of the cement weight. This dosage of superplasticizer is considered the saturation dosage for cement paste. Saturation dosage itself is defined as the dosage beyond which there is no increase or change to the slump flow spread of mixes. The results of trial mixes of cement paste are shown in section 4.3.

The optimum sand to mortar ratio was determined by trial mixes of the different sand/mortar ratio which ranged from 0.45 to 0.5. According to many studies (EFNARC 2002, Domone 2006, Safiudin 2008), it was possible to determine the optimum ratio. Trial mixes were for 4 litres of mortar so that the mix proportions were based on the volumetric proportions. The optimum ratio was determined according to the flowability and strength aspect. The optimum ratio was 0.47. The mix proportions of mortar with different sand mortar ratios are shown below in Table 3.3.

Table 3.3: Mortar mix proportions (4 litres)

S/M	W/P	Water (Kg)	Cement (Kg)	Sand (Kg)
0.50	0.32	0.993	3.102	5.28
0.49	0.32	1.013	3.166	5.17
0.48	0.32	1.033	3.23	5.07
0.47	0.32	1.053	3.29	4.96
0.45	0.32	1.094	3.42	4.75

In the mortar mixtures, there were two types of replacement i.e. 5%, 10%, 15% and 20% of cement was replaced by fly ash and metakaolin, whilst in other mixes 5%, 10%, 15% and 20% of sand was replaced by limestone and kaolin. In addition to this, there was a control mixture which consisted of using only cement as cementitious material and sand as fine aggregate. All mortar mixtures are described in Table 3.4 and the mix proportion of each mortar mixtures is shown in Table 3.5.

The trial concrete mixes with different sand /mortar and coarse aggregate volumes with 19mm graded aggregate was conducted for determining the optimum coarse aggregate volume in concrete depending on the workability and strength. The details of concrete mixes are described in the Table 3.6. The optimum volume of coarse aggregate was 0.33 depending on the results of workability and strength of concrete which are shown in sections 4.6.2 and 4.7.2. In order to study the effect of coarse aggregate volume on the behavior of HSSCC, four mix proportions were used for this purpose and are shown in Table 3.6. The results of workability were determined depending upon the flowability and passing ability and segregation resistance, while the results of the hardened state based on compressive strength. The shaded mixtures presented the control concrete mixtures with different coarse aggregate volume at the same mortar mix proportions.

Table 3.4: Details of mortar mixes

No.	Symbol	Powder type	Replacement level (%)	Type of replacement
1	C	Cement	0	-
2	FA5	Fly ash	5	Cement
3	FA10	Fly ash	10	Cement
4	FA15	Fly ash	15	Cement
5	FA20	Fly ash	20	Cement
6	MK5	Metakaolin	5	Cement
7	MK10	Metakaolin	10	Cement
8	MK15	Metakaolin	15	Cement
9	MK20	Metakaolin	20	Cement
10	LSP5	Limestone	5	Sand
11	LSP10	Limestone	10	Sand
12	LSP15	Limestone	15	Sand
13	LSP20	Limestone	20	Sand
14	K5	Kaolin	5	Sand
15	K10	Kaolin	10	Sand
16	K15	Kaolin	15	Sand
17	K20	Kaolin	20	Sand

Table 3.5: Proportions of mortar mixes (4 L of mortar)

Symbol	C (Kg)	FA (Kg)	MK (Kg)	S (Kg)	LSP (Kg)	K (Kg)	w/p	SPD (% of powder weight)
C	3.29	0	0	4.94	0	0	0.32	1.1
FA5	3.125	0.165	0	4.92	0	0	0.32	0.8
FA10	2.962	0.329	0	4.85	0	0	0.32	0.7
FA15	2.797	0.494	0	4.80	0	0	0.32	0.65
FA20	2.632	0.658	0	4.75	0	0	0.32	0.6
MK5	3.125	0	0.165	4.94	0	0	0.32	1.6
MK10	2.962	0	0.329	4.90	0	0	0.32	2
MK15	2.797	0	0.494	4.87	0	0	0.32	2.5
MK20	2.632	0	0.658	4.85	0	0	0.32	-
LSP5	3.29	0	0	4.69	0.247	0	0.32	1.2
LSP10	3.29	0	0	4.44	0.494	0	0.32	1.4
LSP15	3.29	0	0	4.19	0.741	0	0.32	1.6
LSP20	3.29	0	0	3.95	0.988	0	0.32	1.4
K5	3.29	0	0	4.69	0	0.247	0.32	1.3
K10	3.29	0	0	4.44	0	0.494	0.32	1.8
K15	3.29	0	0	4.19	0	0.741	0.32	2.3
K20	3.29	0	0	3.95	0	0.988	0.32	-

No.	Symbol	Description
1	C	Cement
2	FA	Fly ash
3	MK	Metakaolin
4	S	Sand
5	LSP	Limestone powder
6	K	Kaolin
7	W/P	Water powder ratio
8	SPD	Superplasticizer dosage

Table 3.6: Trial mixes of concrete

Vol. ratio of C. Agg.	Coarse Agg. Vol.*(by weight•)	S/M	Fine Agg. Vol.*(by weight•)	Paste Vol.*	Cement Vol.* (by weight•)	Water (by weight•)
0.29	0.29(757)	0.50	0.355(937)	0.355	0.167(520)	166.7
0.29	0.29(757)	0.49	0.347(918)	0.363	0.172(533)	171
0.29	0.29(757)	0.48	0.34(900)	0.37	0.175(544)	174
0.29	0.29(757)	0.47	0.334(881)	0.376	0.178(554)	177
0.29	0.29(757)	0.45	0.32(843)	0.39	0.185(575)	184
0.31	0.31(809)	0.50	0.345(911)	0.345	0.163(505)	162
0.31	0.31(809)	0.49	0.338(892)	0.352	0.166(516)	165
0.31	0.31(809)	0.48	0.331(874)	0.359	0.169(527)	168
0.31	0.31(809)	0.47	0.324(856)	0.366	0.173(538)	172
0.31	0.31(809)	0.45	0.311(820)	0.379	0.179(558)	178
0.33	0.33(861)	0.5	0.335(884)	0.335	0.157(490)	156
0.33	0.33(861)	0.49	0.328(867)	0.342	0.161(500)	160
0.33	0.33(861)	0.48	0.322(849)	0.348	0.164(510)	163
0.33	0.33(861)	0.47	0.315(832)	0.355	0.167(520)	167
0.33	0.33(861)	0.45	0.301(796)	0.369	0.174(542)	173
0.35	0.35(914)	0.5	0.325(858)	0.325	0.152(474)	152
0.35	0.35(914)	0.49	0.319(841)	0.331	0.155(484)	155
0.35	0.35(914)	0.48	0.312(824)	0.338	0.159(495)	158
0.35	0.35(914)	0.47	0.306(807)	0.345	0.162(505)	162
0.35	0.35(914)	0.45	0.293(772)	0.357	0.168(525)	168

*Volume of constituent's m^3/m^3 of concrete

•the weight (Kg/m^3)

3.3.4 Mix proportions of concrete mixes

After choosing the proper mix proportions of concrete mixes with 19mm graded coarse aggregate, the mix proportions were fixed in order to study the effect of coarse aggregate properties on the engineering properties of HSSCC in fresh and hardened states with three types of mineral admixtures. The details of the mixes are described in the Table 3.7 and the mix proportions are shown in Table 3.8.

3.3.5 Criteria for evaluating of mix proportions

In order to obtain good properties of fresh and hardened concrete, it is necessary to compare the design procedures with the limited criteria in the previous studies in order to avoid the detrimental effect of mix proportions on the properties of concrete. Firstly, the w/b ratio varies according to the requirement of design. The limited range of w/b ratio of high performance concrete is between 0.28-0.38 according to Ma and Dietz (2002) or between 0.22-0.40 by weight for high strength concrete as mentioned in Abdur Rashid and Abul Mansur (2009) study or between 0.8-1.1 by volume of SCC according to EFNARC specification (EFNARC 2002). Koehler & Fowler (2007) recommended that w/b of SCC is between 0.3-0.45; while Safiuddin (2008) concluded that the range of w/b ratio with which to obtain compressive strength from 50 to 110 MPa of HPSCC is between 0.24 to 0.42. In light of this, the ratio used in this study was 0.32 and was in the range of high strength self compacting concrete. Secondly, depending on studies, binder content in the mixes varies with the objective of the study. Binder content includes either cement alone or cement with other types of powder whether pozzolan or filler. The range of total binder in SCC was between 400 - 600 Kg/m^3 according to EFNARC (2002).

CIP33 (2001) reported that the total binder content with which to obtain high strength concrete was between 415 - 650 Kg/m^3. After analysing the data of SCC mixes for 11 years, Domone (2006) reported that the binder content of SCC mixes was between 425-625 kg/m^3. The total binder of HSC and HPC was between 390-560 Kg/m^3 according to (ACI 363R-92, 2005; Gutiérrez & Cánovas 1996, cited by Safiuddin 2008). In addition, Xie et al (2002) reported the binder content of HSSCC should exceed 500 Kg/m^3. The binder content in this study was 520 Kg/m^3 and was in the range of HSSCC.

Table 3.7: Details of concrete mixes

Symbol	Replacement level (%)	Replacement type	Max. size of coarse agg. (mm)	Type of coarse agg. grading	Flakiness ratio in coarse agg.
CC20	100	Cement	19	Continuous	low
CG20	100	Cement	19	Gap	low
CC14	100	Cement	14	Continuous	low
CS14	100	Cement	14	Single	low
CS10	100	Cement	10	Single	low
CS10HFR	100	Cement	10	Single	High
FA10C20	10	Cement	19	Continuous	low
FA10G20	10	Cement	19	Gap	low
FA10C14	10	Cement	14	Continuous	low
FA10S14	10	Cement	14	Single	low
FA10S10	10	Cement	10	Single	low
FA10S10HFR	10	Cement	10	Single	High
MK10C20	10	Cement	19	Continuous	low
MK10G20	10	Cement	19	Gap	low
MK10C14	10	Cement	14	Continuous	low
MK10S14	10	Cement	14	Single	low
MK10S10	10	Cement	10	Single	low
LP20C20	20	Sand	19	Continuous	low
LP20G20	20	Sand	19	Gap	low
LP20C14	20	Sand	14	Continuous	low
LP20S14	20	Sand	14	Single	low
LP20S10	20	Sand	10	Single	low

Table 3.8: Mix proportion of concrete mixes

Symbol	w/b	C (Kg/m^3)	FA (Kg/m^3)	MK (Kg/m^3)	F.Agg. (Kg/m^3)	LP (Kg/m^3)	C.Agg. (Kg/m^3)	SPD (% of powder weight)	VMA (% of powder weight)
CC20	0.32	520	-	-	832	-	861	1.1	-
CG20	0.32	520	-	-	832	-	861	1.1	0.3
CC14	0.32	520	-	-	832	-	861	1.1	-
CS14	0.32	520	-	-	832	-	861	1.1	0.2
CS10	0.32	520	-	-	832	-	861	1.1	-
CS10HFR	0.32	520	-	-	832	-	861	1.1	0.4
FA10C20	0.32	468	52	-	824	-	853	0.7	-
FA10G20	0.32	468	52	-	824	-	853	0.7	-
FA10C14	0.32	468	52	-	824	-	853	0.7	-
FA10S14	0.32	468	52	-	824	-	853	0.7	-
FA10S10	0.32	468	52	-	824	-	853	0.7	-
FA10S10HFR	0.32	468	52	-	824	-	853	0.7	-
MK10C20	0.32	468	-	52	828	-	859	2	-
MK10G20	0.32	468	-	52	828	-	859	2	-
MK10C14	0.32	468	-	52	828	-	859	2	-
MK10S14	0.32	468	-	52	828	-	859	2	-
MK10S10	0.32	468	-	52	828	-	859	2	-
LP20C20	0.32	520	-	-	666	166	866	1.4	-
LP20G20	0.32	520	-	-	666	166	866	1.4	-
LP20C14	0.32	520	-	-	666	166	866	1.4	-
LP20S14	0.32	520	-	-	666	166	866	1.4	-
LP20S10	0.32	520	-	-	666	166	866	1.4	-

Thirdly, paste volume should be more than the voids volume between packed aggregate (coarse and fine aggregate) to act as a liquid transport for the aggregate particles and to provide good flowability and segregation resistance. The range of paste volume of SCC was between 28-40% of concrete volume according to Koehler & Fowler (2007), ACI Committee 211H (2006) and the European Guidelines of SCC (2005). Safiuddin (2008) concluded that the range of paste volume of SCHPC was between 0.24-0.42 m^3/ m^3 in order to give slump flow ranges between 550-850mm. The paste volume for this study was 0.355 m^3/m^3 and is considered to be in the range of HSSCC mixes.

Fine aggregate/ mortar ratio plays a crucial role in affecting the workability and strength of mixes. With this in mind, many studies have determined this ratio in the limited range in order to control the properties of the mixes. EFNARC (2002) specified the optimal range of sand to mortar ratio in mix design finding that it was between 40-50% depending on the properties of paste and assuming a sand grade of between 4.74-0.125mm. After analysing the data of SCC mixes for 11 years, Domone (2006) reported that these studies specified the sand mortar ratio as ranging between 38-54% whilst 80% of these studies determined this ratio to range between 41-52%. Finally, Domone (2006) concluded that the optimum ratio was 47.5%. In the report regarding SCC mix design (UCL method) in 2009, Domone reported in the calculation of mix design method that this ratio was 45%. In the present study, this ratio used was 47% and was inside the range used by most of the studies.

The determining of fine aggregate/ total aggregate by weight depends on the grading and shape of both of coarse and fine aggregate. Fine aggregate/ total aggregate ratios by weight are determined according to the ratios employed in various past studies (Xie et al 2002, the European Guidelines of SCC 2005, Domone 2006, and Koehler & Fowler 2007) namely between 0.4-0.55. In the present study this ratio was 0.491.

Coarse aggregate content is considered an important factor in affecting passing ability, segregation resistance and strength of concrete. After analysing the data of SCC mixes for 11 years Domone (2006) reported that 80% of these studies used the range of 29.1-34.8% which is equivalent to 770-925 kg/m^3 for aggregate with specific gravity of 2.65. EFNARC 2002 determined that the content of coarse aggregate was between 0.28-0.35. Therefore, in this study the trial mixes for determining the optimum content were between 0.29-0.35 whilst the optimum content of 0.33 was inside the range of these studies.

Finally, all these values were fixed so as to compare between the effect of coarse aggregate characteristics on the fresh and hardened properties of HSSCC.

3.4 Mixing procedures of mortar

The mixing process was achieved by using a bowl mixer which has a capacity of 5L as shown in Figure 3.2. Fine aggregate was initially added followed by cement and other mineral admixtures (fly ash, metakaolin, limestone and kaolin). These were mixed in dry state for 2 minutes before two thirds of water was slowly poured into the mix followed by another third of water which was mixed with specified superplasticizer dosages (which were determined by trial mixes). The mixing was continued for a total of 5 min. For each mix, 18 specimens were cast in steel moulds with dimensions of 50×50×50 mm without any compaction. They were covered by a polythene sheet to prevent water loss and were demoulded after 24 hours before being immersed in the curing water until the time of the test. Three specimens were prepared for every age and were tested at 3, 7, 14, 28, 60 and 90 days for compressive strength test and at 28 days for water absorption test.

3.5 Mixing Procedures of concrete

Mixing procedure has a great affect on workability and homogeneity and thus the strength of the mix. The mixer used in this study was a pan mixer which has a capacity of 0.1 m^3 as shown in Figure 3.2. Before beginning the mixing, the mixer should be kept clean, moist and free from excess water. The mixing procedure is summarized as follows. Firstly, the sand should be placed with third of the water and mixed it for 1 ½ minutes. Secondly, the powder (cement+ mineral admixture) should be placed with another third of the water and mixed it for 2 ½ minutes. Thirdly, granite stone should be placed with the last third of water which was mixed with two thirds of superplasticizer and mixed for 2 minutes and then the mixture was left for 2 minutes to rest. Fourthly, the remaining third of the superplasticizer was added and then mixed into the mixture for 2 minutes. Finally, the mixture was ready for discharge, test and cast in the mold. The molds were covered by a polythene sheet to prevent the water loss and were demolded after 24 hours before being immersed in the curing water until the time of the test. Three specimens were prepared for every age of test.

3.6 Fresh mortar tests

Four types of mortar tests were used in this research. Two of these tests, namely the mini slump flow and V-funnel test, are commonly used to test for self compactibility of self compacting mortar. The third test is known as the setting time test and finally the fourth one is bleeding test. These tests for high strength self compacting mortar will be explained in the following paragraphs.

3.6.1 Mini slump flow

This kind of test is described by EFNARC (2002) and as shown in Figure 3.3. The truncated cone mould is used to conduct this test. The truncated cone mould was placed on a level plate, and filled with mortar and then lifted upward. After lifting the mould, mortar would spread on the plate forming approximately a circular shape. The perpendicular diameters of this circle would be measured. The final diameter would be the average of these two diameters (d1 and d2) and as shown in Figure 3.3.

Figure 3.2: Bowl mixer for mortar (left) and pan mixer for concrete (right)

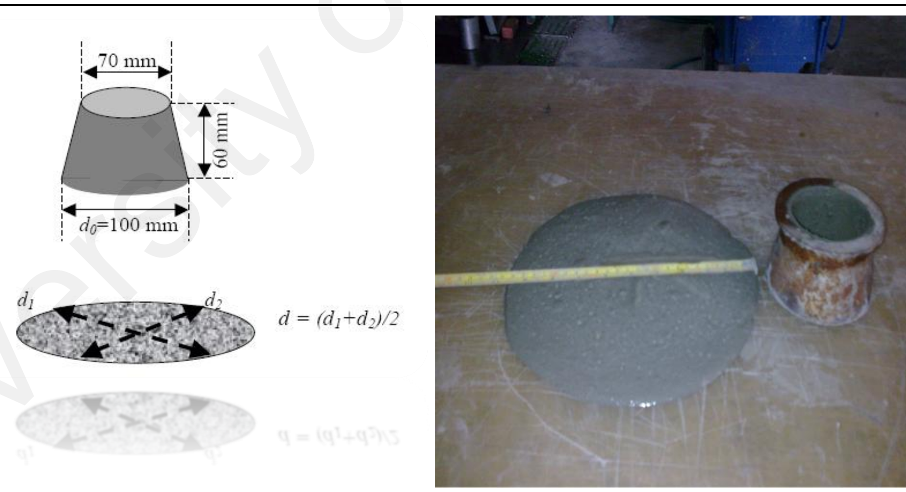


Figure 3.3: Mini slump cone for mortar

3.6.2 Mini V-funnel flow

This test is also described by EFNARC (2002) and is shown in Figure 3.4. Before the V-funnel is filled with mortar, the bottom orifice should be closed. After filled with mortar, the bottom outlet should be opened to allow for mortar to flow through the orifice. The time of flow is calculated from the time of opening the bottom outlet until the light is visible from the top.

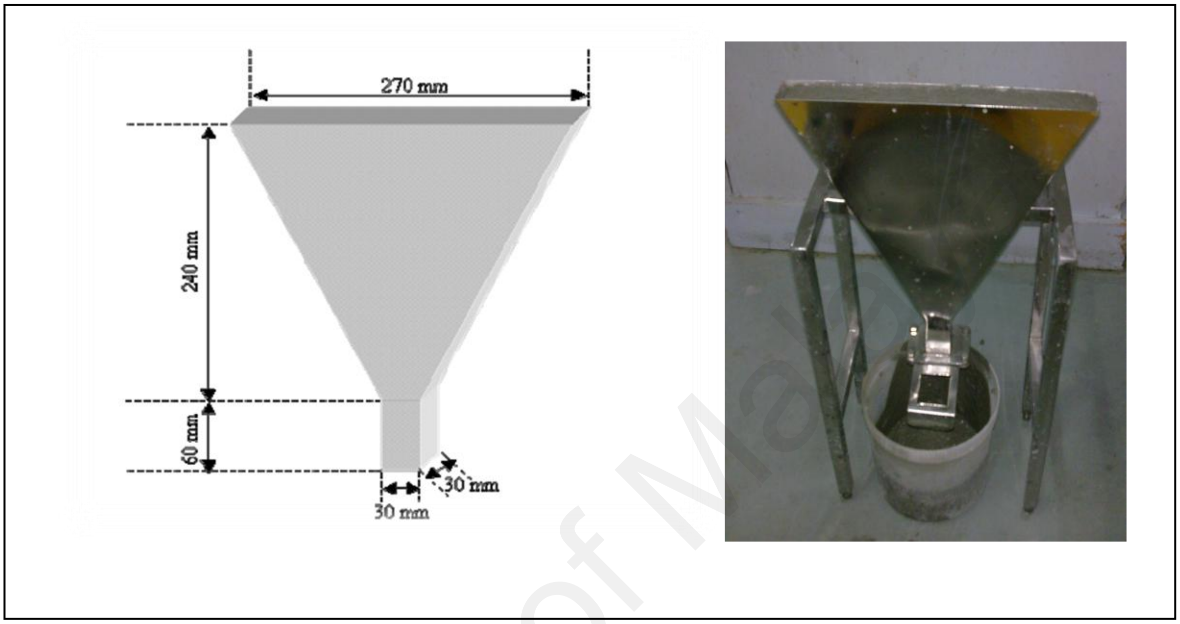


Figure 3.4: Mini V-funnel

3.6.3 Setting time

The setting time of the mortar mixes were determined according to ASTM C 403 (1995). The test is performed by measuring the force required to make the needle penetrate the mortar to a depth of 25 mm. Initial and final setting times are the times when the force required for penetration give values of 3.5 and 27.6 MPa, respectively.

3.6.4 Bleeding

This test was used to evaluate the amount of mixing water which bleeds from SCC mixes. The procedures for conducting this test were followed in accordance with ASTM C232 and described below.

The fresh concrete was casted in a cylinder container with inside diameter 255±5 mm and height of 280 ±5mm and then trowels the surface of mortar. The cylinder was placed on the level ground without being exposed to any vibration. However, the cylinder was covered to prevent evaporation of water for the entire duration of the test time except when withdrawing it from the water. Withdrawing the water was done by using a pipette and started to withdraw the water after 40 minutes from cast time in cylinder and continue for withdrawal of water for 10 minutes. In order to ease the accumulation of water, tilt the sample carefully placing a block so that it was lifted approximately 50 mm from one side. After removal of the water from the mortar surface, the container was reverted as it was in a level position. The water removed from the surface was placed into the 100 ml graduated cylinder and the accumulated water in the graduated cylinder was measured after each withdrawal. This process came to an end only when the total quantity of bleeding water was desired. Filter the accumulated bleeding water in the cylinder by using a piece of cloth. Finally, the bleeding water was calculated by volume per surface unit area as follows:

$$V=V_l/A \tag{3.1}$$

where:

V_1 = volume of bleeding water computed during the chosen time period, ml,

A = area of exposed mortar, cm^2.

The relative rate of bleeding was specified as the test advances by comparing the volume of bleeding water for each equivalent time period. The accumulated bleeding water can be expressed as a percentage of the pure mixing water of the test sample, as follows:

$$C = (w/W) \times S \tag{3.2}$$

$$\text{Bleeding rate} = (D/C) \times 100 \tag{3.3}$$

where:

C = weight of the water in the test sample, g,

W = total weight of the batch, kg,

w = net mixing water (the total amount of water minus the water absorbed by the aggregates), kg,

S = weight of the sample, g, and

D = weight of the bleeding water, g, or total quantity taken from the test sample in cubic centimetres multiplied by 1 g/cm^3.

3.7 Hardened mortar tests

Two types of hardened mortar tests were used in this research. The first one test is known as the compressive strength test and second one is known as water absorption test. These tests for high strength self compacting mortar will be explained in the following paragraphs.

3.7.1 Compressive strength

Compressive strength of 50 mm cube mortars was determined at 3, 7, 14, 28, 56 and 90 days of mortar age. The test was conducted in accordance with British Standard (BS 1881: 1983c) using a universal testing machine made in British with 2000KN capacity.

3.7.2 Absorption

The water absorption of mortar 50 mm cubes were tested after 28 days of water curing and according to BSI 1881 1983i. Specimen used was placed in an oven with temperature

105±5ºc for 72 hours. The specimen was then weighed before being immersed in water with 25±5 mm of water on top and the excess water was removed by using a paper towel. These samples were then weighed and the weight was taken as the oven-dried mass. Immediately after that it was submerged in water for 30 ±0.5 minutes at a depth such that the water was 25 ±5 mm above the specimen. It was then brought out of the water and the surplus water was removed by using paper towel. The weight was taken instantly and was recorded as the immersed mass.

3.8 Fresh concrete tests

Six types of concrete tests were used in this study to evaluate fresh concrete. These tests for high strength self compacting concrete will be explained in the following paragraphs.

3.8.1 Slump flow test

This test was most widely used for estimating the flowability of SCC. This test was conducted according to EFNARC (2002). The instruments for conducting this test are slump cone and a marked rectangular table as shown in Figure 3.5. The procedures necessary for achieving this test were mentioned with the following paragraphs. The required amount of concrete for conducting this test is at least 6 litres. The slump cone was placed on the level base plate so that it is perpendicular on the table. The table used was marked with concentrated circles with diameter of 500mm. This table must be placed on horizontal level. This table was cleaned with water and then wiped with a piece of cloth so as to ensure that the table is moist but without excess water. The centre of the slump cone was placed so that it was corresponds with the centre of the circle. The cone must be fixed well to the base plate. The whole cone was filled with concrete so that the excess concrete was removed from the top of the cone by using a trowel and without damping. Following this is lifting the cone vertically and instantly. The time was constantly monitored in order to establish when the concrete reaches the border of the 500mm circle. This is known as the T_{500mm} time. After stopping the flow of concrete, the flow diameter was measured and it is slump flow diameter (SFD) (mm).

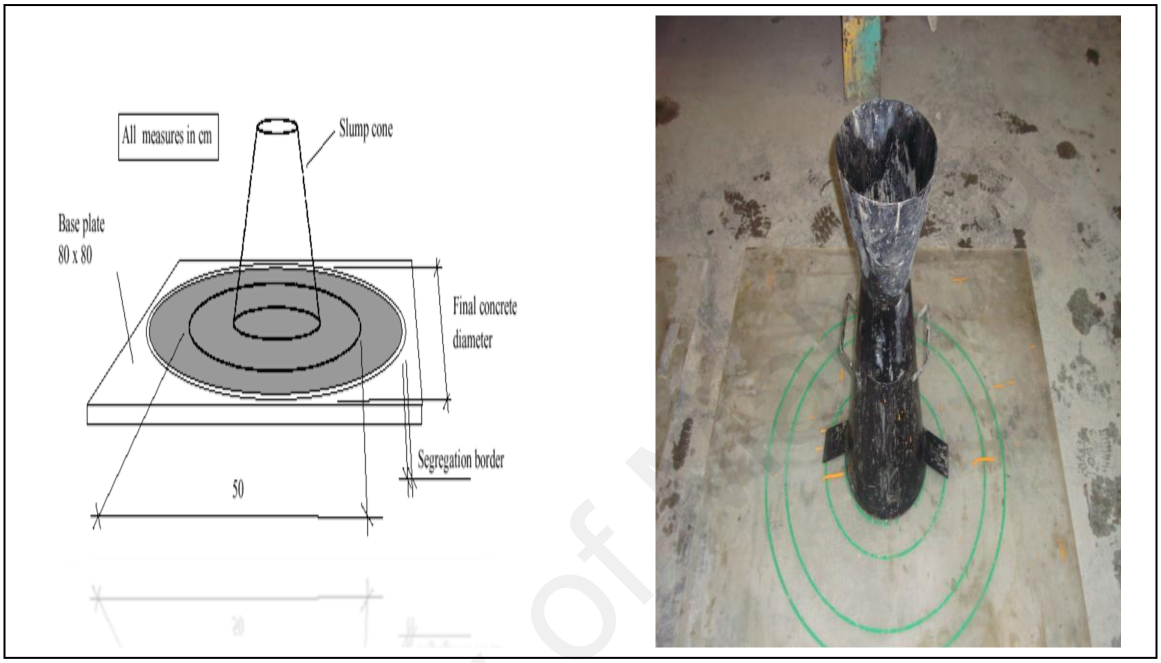


Figure 3.5: Slump Flow Test

3.8.2 V-Funnel test

This test is used to evaluate the flowability of SCC and is thought to estimate the segregation resistance of a mixture. This test was conducted according to EFNARC. The instruments used in this test are V-funnel box and stop watch and are shown in Figure 3.6. The required procedures for achieving this test are briefly summarised as the follows. The required amount of concrete for conducting this test is not less than 12 litres. V-funnel box was placed at horizontal level. The inner faces of the box was moistened and free from excess water while the bottom outlet of this box was not closed in order to facilitate the removal of surplus water. The bottom outlet was closed before filling the box with concrete. The filling period was not exceeded 10 minutes. The measuring of time was started as soon as the bottom outlet was opened and was stopped as soon as the light was seen through the box. The recorded time presents Flow time (f_t). After 5 minutes, the same process was repeated and the recorded present time flow is presented after 5 minutes (f_{t5min}).

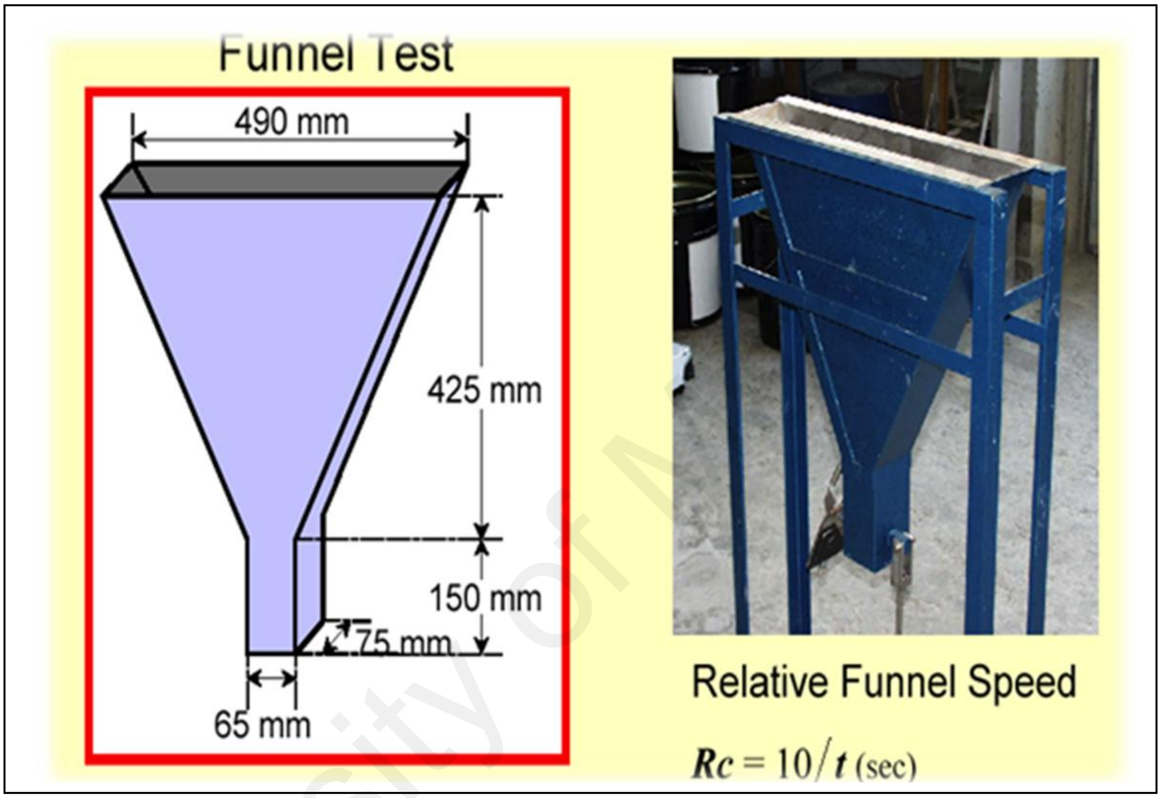


Figure 3.6: V-Funnel test

3.8.3 J-Ring test

This test is used to evaluate the passing ability of concrete. The test was first introduced in Japan and later developed at the University of Paisley, UK. The test is conducted to determine the passing ability of the concrete and according to EFNARC. The instruments used in this test are open steel ring with a rectangular section of 30mm x 25mm as well as a marked base plate with a slump cone as shown in Figure 3.7. The required procedures for achieving this test are briefly detailed as follows. The required amount of concrete for conducting this test is at least 6 litres. The base plate and cone was cleaned with water and then wiped with a piece of cloth to ensure that this table was moistened but free from surplus water. The slump cone and J-ring were placed on the level base plate so that it is perpendicular to the base plate and the slump cone was inside the ring so that the centre of the cone corresponds with the centre of the J-ring. It is also important to note that the cone was fixed firmly onto the base plate. The whole cone was filled with concrete so that the excess concrete was removed from the top of the cone by using a trowel and without damping. Following this, lifting the cone vertically because this should cause fresh concrete to flow out of the cone easily passing through the bars of the ring and then the difference in height of concrete inside and outside the ring was measured and taken the average of four locations (mm). Finally, observing whether there was any mortar or cement paste at the edge of the fresh concrete circle on the table so that it was not observed any coarse aggregate at the edge.

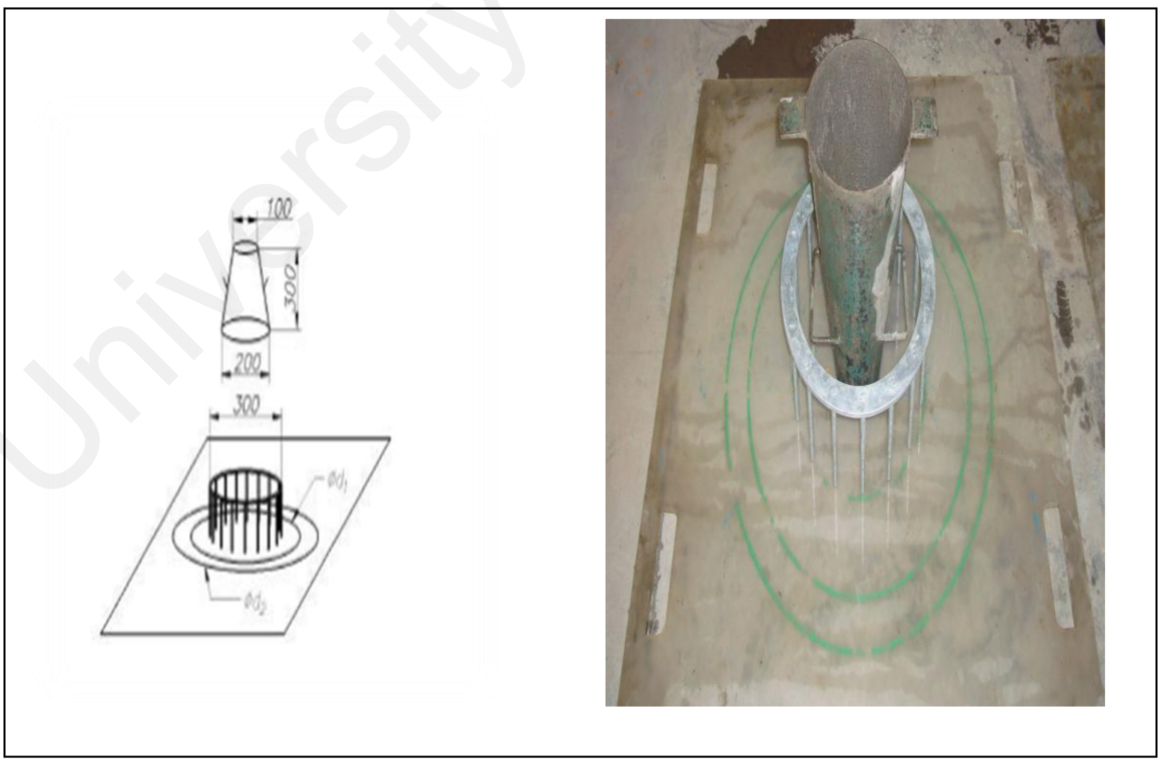


Figure 3.7: J-ring test

3.8.4 L-Box Test

This test is used to evaluate the passing ability of SCC and according to EFNARC. This test consists of an L-box which comprised of vertical and horizontal parts. The concrete passes from the vertical part to the horizontal part through a steel bar and all dimensions are indicated in Figure 3.8. The required procedures for conducting this test are described in the next sentences. The amount of concrete required to achieve this test is approximately 14 litres. The box was placed on the ground at horizontal level. The concrete was initially cast in vertical part whilst the gate between the vertical and horizontal part was closed. In order to assess whether any segregation occurs, the concrete in this part was left to rest for one minute. Following this the gate was opened to allow the concrete in the vertical part to flow out through steel bars to the horizontal part. After the concrete flowing has stopped, the height of concrete on both sides was measured in order to determine the blocking ratio (BR). BR is the ratio of the height of concrete at end of the box (H2) to the height of concrete at the beginning of the box (H1). Accepted (BR) values should range between 0.8-1.00 and high BR values mean high passing ability.

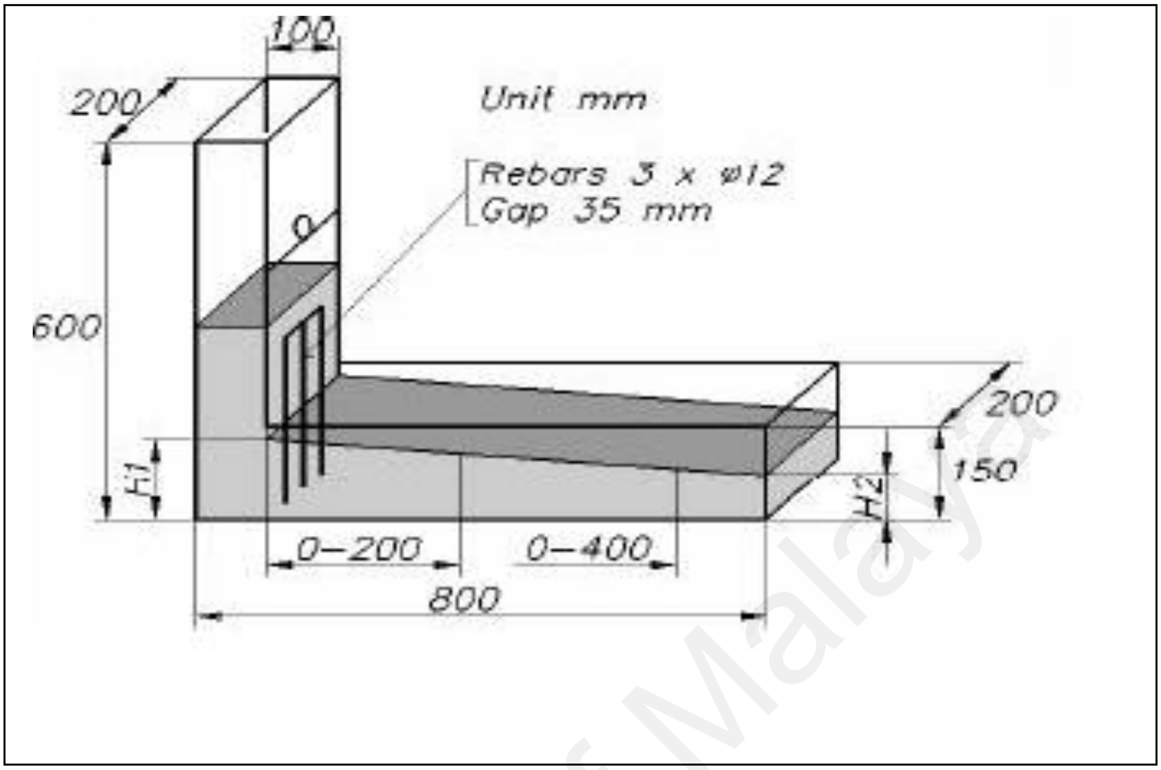


Figure 3.8: L-Box test

3.8.5 GTM screen stability test

This test is used to evaluate the segregation resistance of SCC according to EFNARC standard and involves the use of a 5mm sieve and pan with diameter of 350mm as well as a weight scale as shown in Figure 3.9. The amount of concrete required for conducting this test is approximately 10 litres. This amount was taken in a pail and left for a period of time in order to establish whether or not any internal segregation takes place. The following paragraphs represent the practical procedures for conducting this test.

The concrete was left in the bucket for 15 minutes and covered with a lid in order to avoid any evaporation of water from the concrete. Following this, half of the total amount of concrete was poured into the sieve with its pan with height of 1/2 m. They were then placed on a weight scale and left for two minutes. The mortar which was separated from concrete and passed through the sieve must be determined. The mass of the sieve and pan when it is empty was determined. Inspect the concrete surface for any bleeding water and monitor it constantly. Approximately 5±0.2 Kg of the top concrete was casted in the pail into a pouring container. The mass of the filled container was specified. The mass of the empty container was specified. The full concrete from the container with one smooth movement continuously was casted onto the sieve so that the height of the container from the sieve is around half a meter. The weight of the empty container was specified. Mass of the net concrete poured (Ma) into the sieve was calculated by calculating the difference between the full and empty weight of the sieve pan. It was waited for two minutes to allow for the mortar to separate from the concrete by flowing through the sieve pan. The weight of sample which is passed through the sieve (Mb) was calculated by subtracting the mass of the empty sieve pan from the mass of the filled sieve pan. Finally the segregation ratio was calculated using the following equation:

$$\text{Segregation Index} = (Mb/Ma) \times 100 \tag{3.4}$$

Figure 3.9: Sieve segregation test instruments

3.8.6 Fresh density

This test is performed to evaluate the density of concrete in fresh state and is conducted in accordance with the ASTM C138 (2004). For this test to be effectively conducted a cubed container with dimensions of 100*100*100mm was required. The mass of this container was calculated (Mm). Fresh concrete was poured into the container directly after mixing in a mixer. The weight of the filled container with concrete (Mc) was measured. The density of concrete (D) was calculated by dividing the weight of net concrete by the container volume (Vm) as shown below:

$$D = (M_c - M_m)/V_m \quad (3.5)$$

where:

D = fresh concrete density, (kg/m^3)

M_c = mass of the full concrete container, (Kg)

M_m: = mass of empty container, (Kg)

V_m = the volume of container, (m^3).

3.9 Hardened concrete tests

The tests of hardened self compacting mortar and concrete are conducted using two methods i.e. destructive and non destructive. All samples were cast in the molds without compaction. All together 756 different samples were cast. All samples were demoulded after 24 hours, marked and cured in water at 20° C± 5 until testing age.

3.9.1 Compressive strength test

This test is considered a very important test with regards to evaluating the strength of hardened concrete. This test is conducted in accordance with BS 1881: Part 116 1983c. The specimens used were 100×100×100mm cubes. Testing for concrete samples was conducted at 3, 7, 28, 56, 90, and 180 days. The machine which was used in the tests is the hydraulic type with 2000KN capacity available in the Concrete Laboratory in the Civil Engineering Department, University of Malaya as shown Figure 3.10.

Compressive strength (MPa) was obtained by dividing the maximum applied load (P) by the area of the sample applied to load (A). The average of the strength results of three sample was taken for each age.

Compressive strength $\alpha_c = P/A$ (3.6)

where:

P = applied load on the sample, N,

A = the exposed area of the sample to load mm^2.

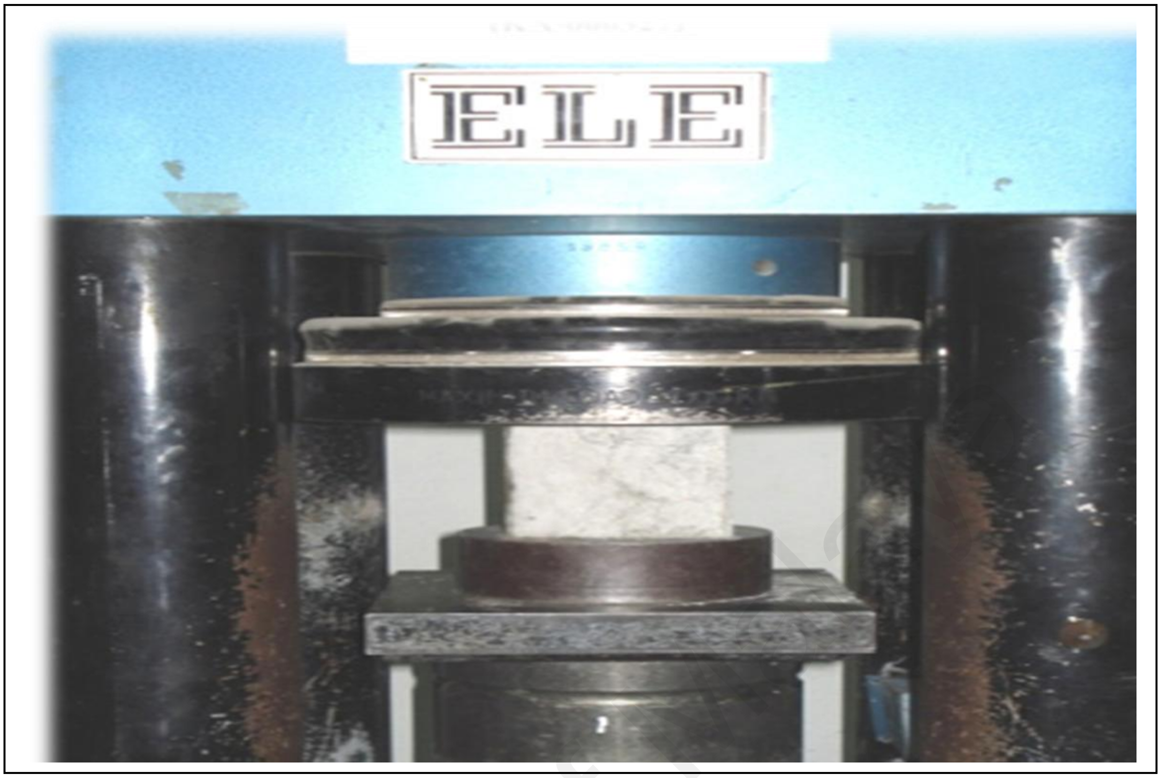


Figure 3.10: Compressive strength test

3.9.2 Splitting tensile strength

The splitting tensile strength was determined at 28 days on cylinders with height of 300mm and diameter of 150mm which were moist cured in water until the date of test according to BS 1881: Part 117 (1983d). The average value of the three specimens for each mix was determined and recorded.

The test was carried out by placing a cylinder specimen horizontally between the loading surfaces of a compression testing machine following which the load was applied until failure of the cylinder, along the vertical diameter as shown Figure 3.11.

The splitting tensile strength was then calculated using the following equation:

$$f_t = \frac{2P}{\pi L D} \qquad (3.7)$$

where:

P = is the applied compressive load, (N)

L = is the cylinder length, (mm) and

D = is the cylinder diameter, (mm).

Figure 3.11: Splitting tensile strength test

3.9.3 Flexural strength test

Flexural strength of concrete was performed on 100x100x400mm prism specimens in accordance with B.S.1881; Part 118 (1983e), using a flexural strength test machine with a capacity of 20000 KN and as shown Figure 3.12.

Since fracture occurs within the central one-third of the beam for all specimens, the modulus of rupture is calculated on the basis of elastic theory, and is therefore equal to:

$$f_r = PL/bd^2 \quad (3.8)$$

where:

f_r = modulus of rupture, (N/mm^2)

P = maximum applied load, (N)

L = span length (mm)

B = width of the specimen, (mm)

d = depth of the specimen, (mm).

Figure: 3.12: Flexural strength test

3.9.4 Static modulus of elasticity

This test was conducted according to BSI (1881: 1983h). The test was conducted on cylindrical specimens with dimensions diameter 150mm×300 mm height which were tested at 28, 90 and 180 days of curing in water. The compressometer i.e. strain measuring gauge was set up axially to the sample and it was put centrally in the compression machine. The strain was recorded when the first stress i.e. basic stress was applied. The basic stress was 0.5 MPa. The stress started to increase steadily by 0.5 MPa/sec until the stress became 1/3 of compressive strength of cylinder concrete and then was kept at this level for 60 sec. The average strain was taken for two loading cycles. The setting up of the apparatus of static modulus is as shown in Figure 3.13.

The cylinder compressive strength (f_{cyl}) was specified depending on the cube strength (f_{cube}) and as shown in the following equation (BSI 1881 1983g):

$$f_{cyl} = 0.85\times f_{cube} \tag{3.9}$$

The static modulus of elasticity E_c (N/mm2) was specified as the average value recorded from two samples by using the following equations (BSI 1881 1983h):

$$\text{Strain Coefficient} = \frac{0.002}{150} \times 0.5 \tag{3.10}$$

$$\mathcal{E} = \text{Gauge division} \times \text{Strain Coefficient} \tag{3.11}$$

$$E_c = \frac{\delta\alpha}{\delta\mathcal{E}} = \frac{\alpha a - \alpha b}{\mathcal{E}a - \mathcal{E}b} \tag{3.12}$$

where:

α_a the upper loading stress= $F_{cyl}/3$ (N/mm^2);

α_b the basic stress (0.5 N/mm^2);

$\mathcal{E}_a$ the mean strain under the upper loading stresses;

$\mathcal{E}_b$ the mean strain under the basic stress.

Figure 3.13: Set up of elastic modulus test

3.9.5 Ultrasonic pulse velocity test

This test was performed according to BS 1881: Part 203 (1986b) by using the Portable Ultrasonic Non-destructive Digital Indicating Tester (PUNDIT). The ultrasonic pulse velocity method consists of calculating the time of travel of an ultrasonic pulse, passing through the concrete. The pulse generator circuit consists of an electronic circuit for generating pulses and a transducer for transforming these electronic pulses into mechanical energy with a vibration frequency. The time of travel between initial beginning and the receiving of the pulse is measured electronically. The path length between transducers, divided by the time of travel, gives the average speed of wave proceeding:

$$V = L / T \qquad (3.13)$$

where:

V Ultrasonic pulse velocity, km/sec,

L Path length, mm and,

T transit time, $\mu\sec$.

3.9.6 Drying shrinkage test

Shrinkage is a time dependent phenomenon, which is measured by instruments with a dial gauge of 0.002 mm accuracy. Measurements of the shrinkage strain are made according to ASTM C157 (1989). After removing the specimens from the mold at an age of 24 hrs they were left exposed to the air until the time of the test. Readings were obtained at ages of 1, 3, 7, 14, 28, 56, 90, 120, 150, 180, 240, 300, and 360 days. Three prisms of (100x100x400 mm) were used for this test and for all ages of tests. The demec gauge with an accuracy of 8µm was manufactured by W.H. Mayers and Son Ltd, England. The demec gauge for each measurement was calibrated by using an invar rod. The reading of shrinkage was determined by taking the average of six readings measured from the two samples.

3.9.7 Absorption test

The water absorption was carried out according to (BSI 1881 1983). Specimens were sured in water until the age of the test. The specimen was placed in an oven at 105±5° for 72 hours and then maintained in an airtight container for more than 24 hours to become cool. The specimen was then weighed before being immersed in water for 25±5 mm of water on top of the excess water which was removed by using a paper towel. The test was performed according to the relevant British Standard. The specimens used were 100 mm cubes and was then weighed and the weight was taken as the oven-dried mass. Instantly after that it was submerged in water for 30 ±0.5 minutes at a depth such that the water was 25 ±5 mm above the specimen. It was then brought out of the water and the surplus water was removed by using paper towel. The weight was taken instantly and was written down as the immersed mass. The correction factor correlated to size is depended on the following formula (BSI 1881 1983g):

$$A= [(Ms-Md)/Md]*100 \tag{3.14}$$

where:

A = Absorption value (%),

M_s = mass of surface dry specimen in air, (kg),

M_d = mass of oven- dry specimen in air, (kg).

3.9.8 Hardened Density

This test was conducted in accordance with BS 1881: part 114. The specimens used were 100*100*100mm concrete cubes. Testing was carried out at 28 days and then the results average of three samples was taken.

Density was obtained by dividing the mass of sample by the volume of the sample. Averaging of the density results of three was taken for each age.

$$D = M/V \tag{3.15}$$

where:

D: density of hardened concrete sample (Kg/m^3)

M: mass of sample (Kg).

V: volume of sample (m^3)

3.10 X-Ray Fluorescence

X-Ray Fluorescence (XRF) is a simple method for the quantitative and qualitative analysis of elemental component in materials. XRF is depended on the photoelectric effect, whereby a specimen radiated via a beam of extremely strenuous photons, will cause a release of an X-ray spectra or secondary fluorescent radiation. Each element has its own set of XRF lines or characteristic release, and thus can be identified and differentiated. The energy and the intensity of these lines are calculated by using a spectrometer.

XRF was used in this study to determine the main elements content (Mg, Si, Na, Al, K, P, Ti, Ca and Fe) of cement, fly ash, kaolin, metakolin and limestone powder. The test was performed at COMBICAT, UM. Specimens were milled to 20 to 30 micrometer prior to analysis. Sample was provided by mixing 0.5 g of sample and 5 g of 110 m spectroflux (Johnson Matthey). The mixture was then put in Pt-Au crucibles and was fired for 10 minutes at 1000˚c in a computerized pellet preparation apparatus. The homogenous fusion was then recast into a glass pellet of 2 mm thickness and 32 mm diameter. Loss of ignition (L.O.I) was computed as the volatile constituent percentage, mainly organic carbon (CO_2) and crystal bound water, driven off from specimens when heated at 1000˚C for an hour. One gram of specimen was used for each report.

3.11 Summary

The experimental program can be summarized in the flow chart shown in Figure 3.14.

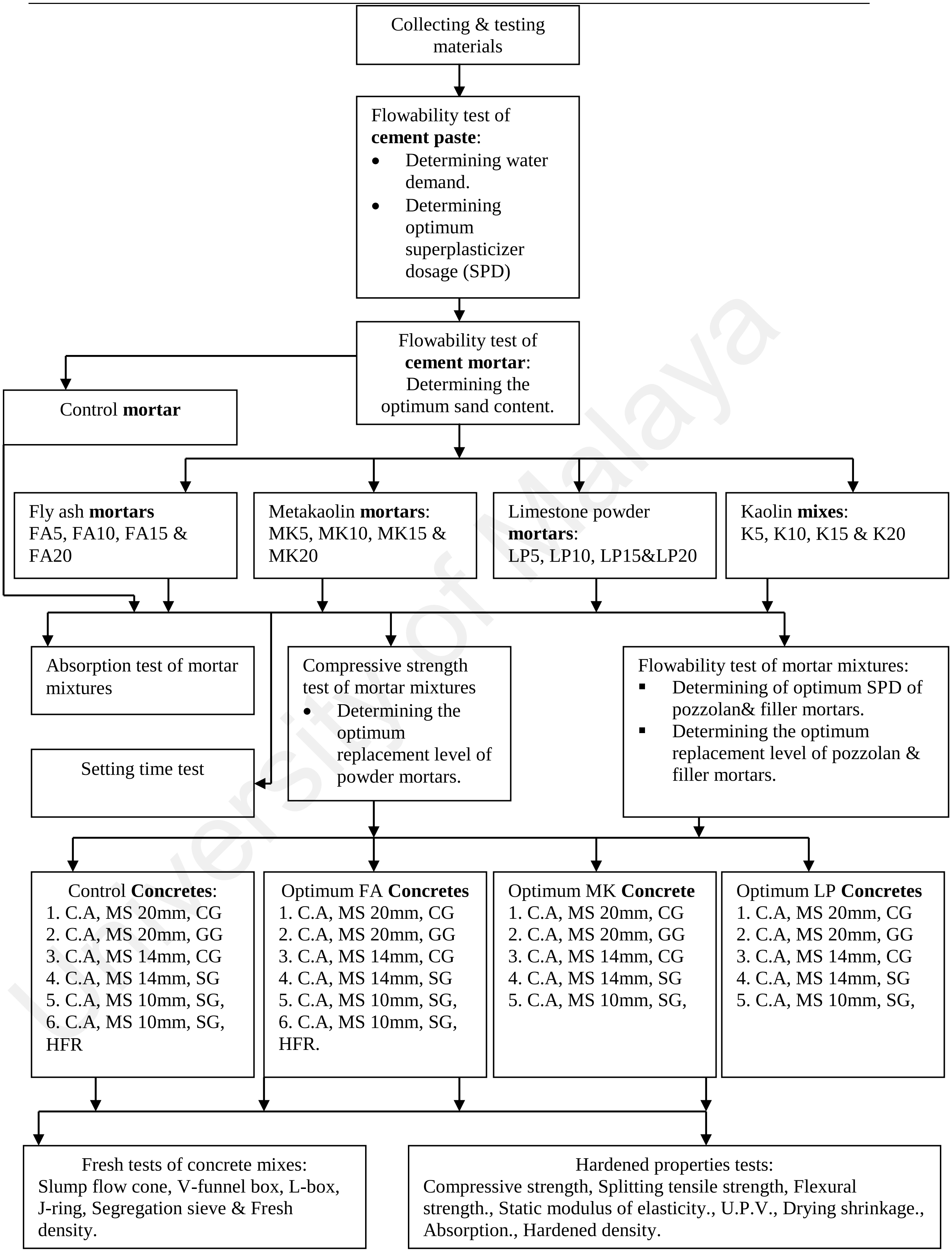


Figure 3.14: Flow chart of experimental program

CHAPTER 4- RESULTS AND DISCUSSION

4.1 Introduction

This chapter presents and discusses the results of the experimental work of material properties and different mixes including paste, mortar and concrete. These mixes have different types and dosage of powders and different coarse aggregate characteristics. The mechanical, durability and time dependent properties of these mixes are evaluated. The discussion is mainly focused on the effect of type and dosage of powder and grading, maximum size and shape of coarse aggregate particles on the properties of concrete.

4.2 Material characterization

4.2.1 Powder

The physical and chemical properties of cement are shown in Table 4.1. The major compounds of cement are computed based on Bogue's equation and comprised of 47.11% tricalcium silicate (C_3S), 30.81% dicalcium silicate (C_2S), 8.87% tricalcium aluminate (C_3A), 9.12% tetracalcium aluminoferrite (C_4AF) and alkalis. It is observed that the Bogue potential composition and loss on ignition (L.O.I) meet limits of Portland cement Type I. The table shows the chemical and physical properties of pozzolanic and filler materials. It is clear that the metakaolin and kaolin has a higher percentage of silica while limestone powder has higher percentage of CaO when compared to other powders, even higher than cement. The specific gravity of all powders is shown in Table 4.1. Cement demonstrates a higher specific gravity compared to other powders. Fly ash has the lowest specific gravity which means that it affects the density of mixes. Figure 4.1 shows the particle size distributions of all powders. Metakaolin and kaolin have finer size distributions. Generally, all particle sizes distribution of pozzolana and filler materials are finer than particle size distribution of cement. Figure 4.2 shows micrographs obtained from scanning electron microscopy on the powders. From the micrographs, the fly ash particles show a spherical shape while kaolin and metakaolin have a plate -like shape while limestone has an irregular shape.

Table 4.1: Chemical and physical properties of powder materials

Powder	Oxides								Specific gravity	Fineness m^2/kg
	Al_2O_3	SiO_2	CaO	Fe_2O_3	MgO	SO_3	K_2O	L.O.I		
Cement	3.72	15.3	72.37	3.33	-	3.77	0.62	1.5	3.11	203
Fly ash	30.7	48.2	8.31	-	-	0.78	1.06	-	2.27	469
niloakateM	41.40	54.5	-	1.15	-	-	2.12	2.52	2.59	753
Limestone powder	-	-	97.42	0.08	2.4	-	-	-	2.72	547
Kaolin	40.40	55.4	-	1.21	-	-	2.9	10.4	2.58	748

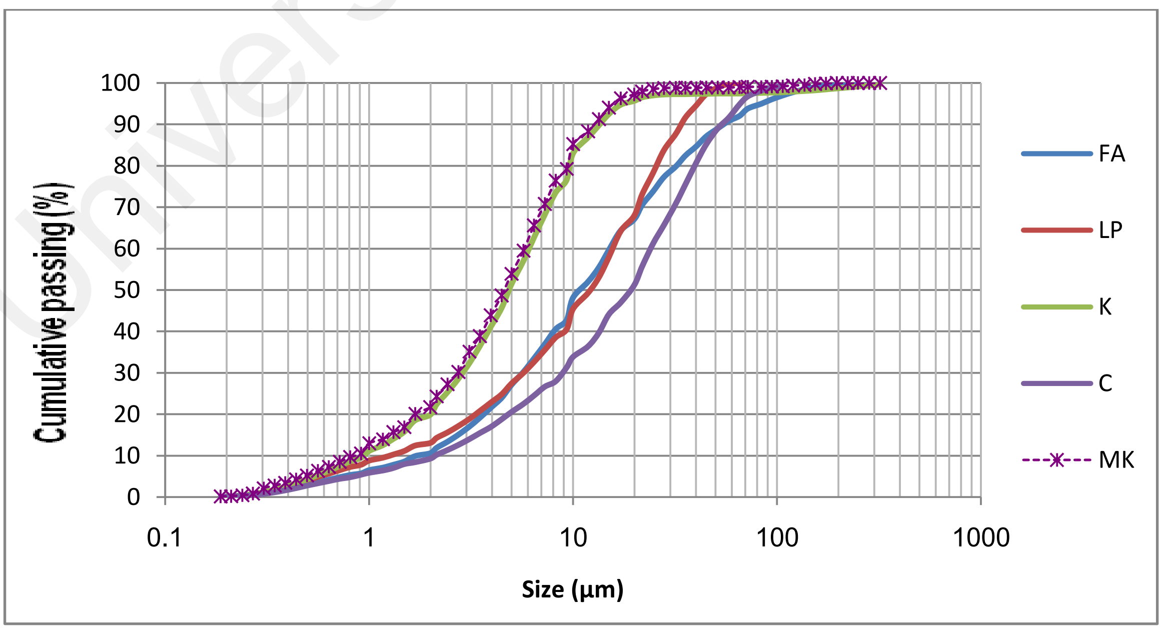
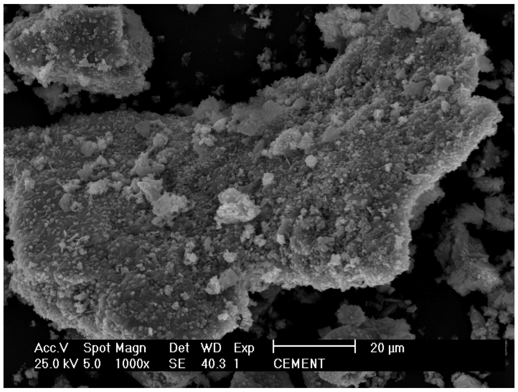
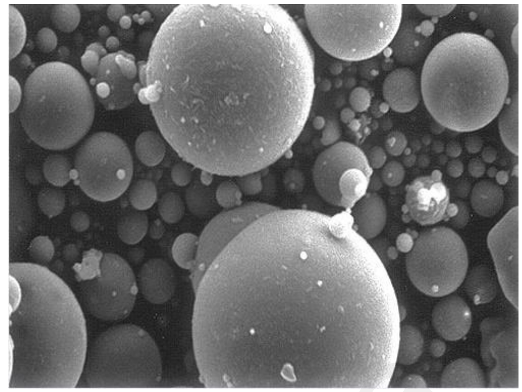


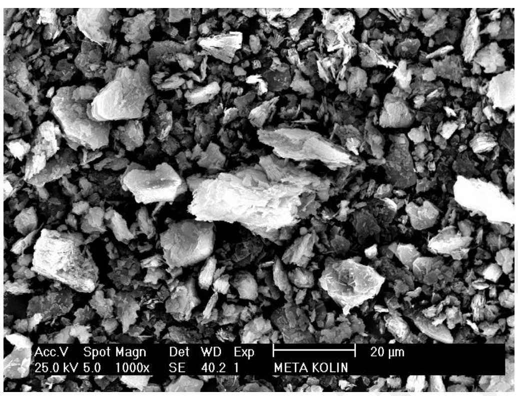
Figure 4.1: Particle size distribution of powders



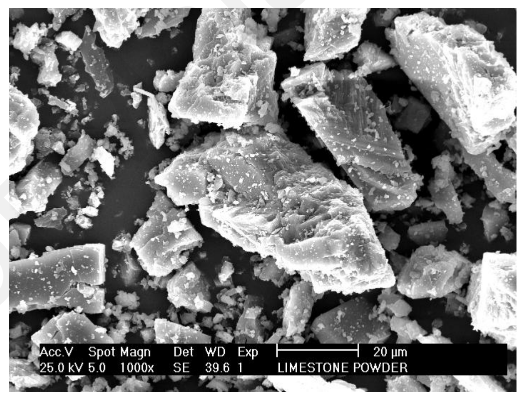
(a) Cement



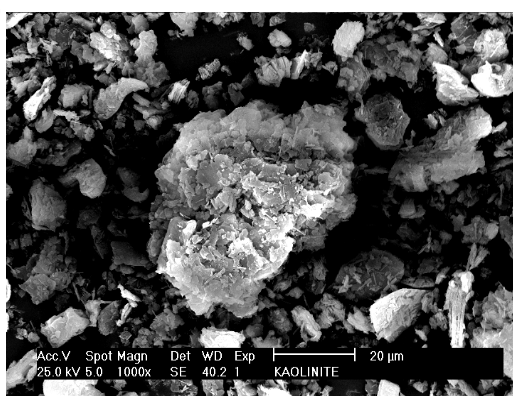
(b) Fly ash



(c) Metakaolin



(d) Limestone



(e) Kaolin

Figure 4.2: Micrographs of powders

4.2.2 Fine Aggregate

The fine aggregate used was silica sand passing 4.75 mm sieve. The apparent specific gravity was 2.64 and water absorption was 0.34%. The sieve analysis of fine aggregates is shown in Figure 4.3 and it conforms to the grading limits of BS 882:1992. The fineness modulus was 2.5.

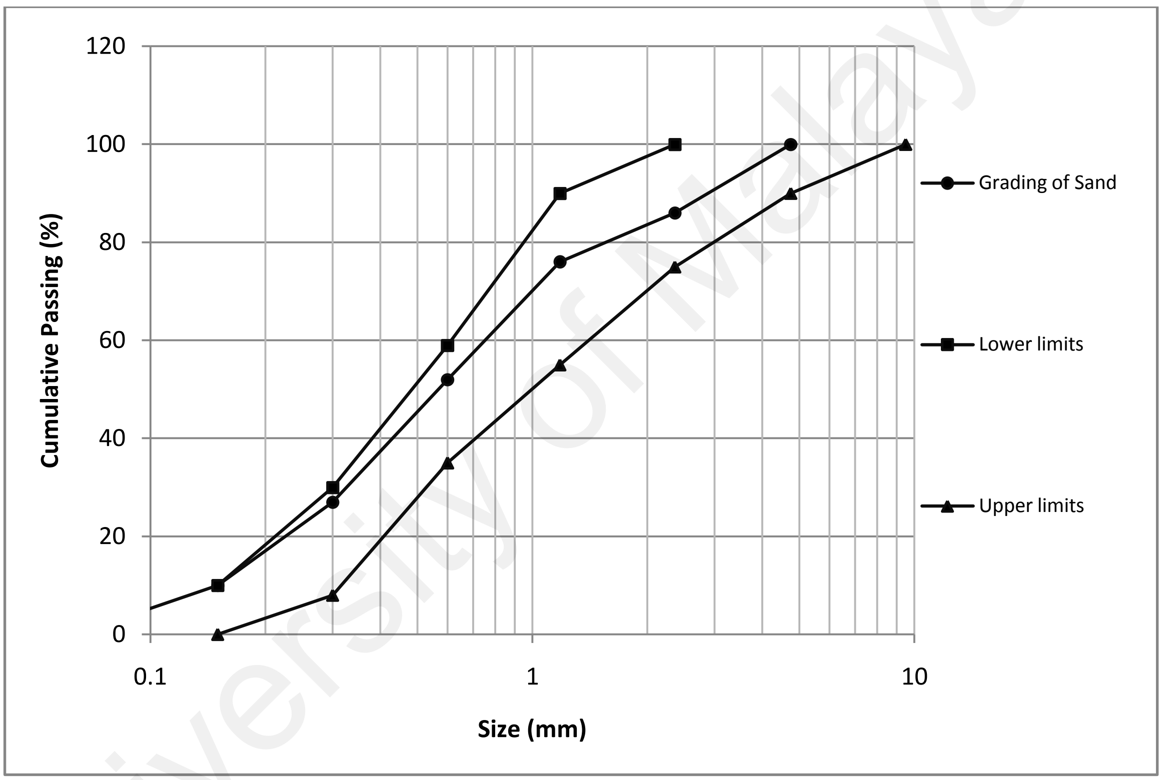


Figure 4.3: Grading for fine aggregate

4.2.3 Coarse Aggregate

The coarse aggregate used was crushed granite with two different shapes i.e. angular as well as a semi rounded and rough surface. Three types of grading and maximum sizes are used namely continuous grade and gap grade of sizes between 19 to 4.75 mm, continuous grade and single grade of sizes between 14.5 to 4.75 mm and single grade of sizes between

9.5 to 4.75 mm. Two types of single sizes which are between 9.5 to 4.75 mm were used, one of them with low flakiness ratio of 3.5% and another one with high flakiness ratio of 47%. The sieve analysis of all categories of coarse aggregates is shown in Table 4.2. The apparent specific gravity of coarse aggregate is 2.61 and water absorption is 0.65%.

Table 4.2: Coarse aggregate grading

No.	Sieve Size	Cumulative passed by weight (%)				
		20mm graded	20mm gap grade	14mm graded	14mm single size	10mm single size
1	19	100	100	100	100	100
2	13.2	58.2	42	97	100	100
3	9.5	32.3	42	48	7	96
4	4.75	0	0	5	0	9
5	2.36	0	0	0	0	0

4.3 Fresh properties of cement paste

The preparation and testing of various binder pastes and the test results for the filling ability of the binder cement pastes with respect to flow spread are presented and discussed in this section. The effects of water-cement (w/c) ratio and high-range water reducer (HRWR) on the filling ability of the binder pastes are discussed. The saturation dosages and water reduction capacity of HRWR as well as the water demand and suitability of cement are highlighted. Finally, this section deduces how the paste filling ability results can be used in the concrete mix design process so that it would abbreviate mix design and saved more materials and hence reduced cost.

Water/ cement (w/c) ratio equivalent to 0.32 by weight was determined, so that this w/c ratio represents the minimum water demand of cement particles to allow the cement paste to initiate flow, where the water is added to cement with different ratios until the cement paste ratio initiate flow in mini slump flow test. This water cement ratio was fixed for all mortar and concrete mixtures as a water cementitious ratio. The optimum superplasticizer dosage was determined based on the results of the mini slump flow cone of cement paste mixes. The figure indicated that as superplasticizer dosage increase, the slump flow diameter will increase. The apparent in Figure 4.4, the optimum dosage is 1.1% of cement weight.

Figure 4.4: Flow spread of cement based paste

4.4 Fresh properties of mortar

The effects of the proportion and the fineness modulus of sand, high-range water reducer (HRWR) dosage, and type of pozzolan and filler powders on flowability and filling ability are presented and discussed. The saturation dosages of the high-range water reducer (HRWR) and the suitability of pozzolan and filler are determined. Finally, this section deduced how the mortar flowing ability results can be used in the concrete mixture design process.

4.4.1 Effect of sand/mortar ratio (S/M)

Figures 4.5 and 4.6 indicate the effect of sand content on the flow ability and filling ability of mortar respectively. The results clearly show that as sand content increases the flow ability of mortar decreases. This could be due to the fact that as the sand content increases the paste layer around the sand particles reduces and thus the friction between the particles is increased thus leading to a decrease in the flow ability. Moreover, the increase of sand content increases the flow time through the mini V-funnel orifice due to blocking of the sand particles and this results from the fact that the distance between the sand particles is closer. These findings are consistent with those of Liu (2009). However, Figure 4.7 indicates the relationship between the bleeding rate and sand content. The increased sand content leads to a high bleeding rate of water as high sand ratio means low cement paste content and thus less possibility for the retention of water.

Hu and Wang (2010) concluded that the increased sand content in the mortar lead to increase viscosity and yield stress of mortar.

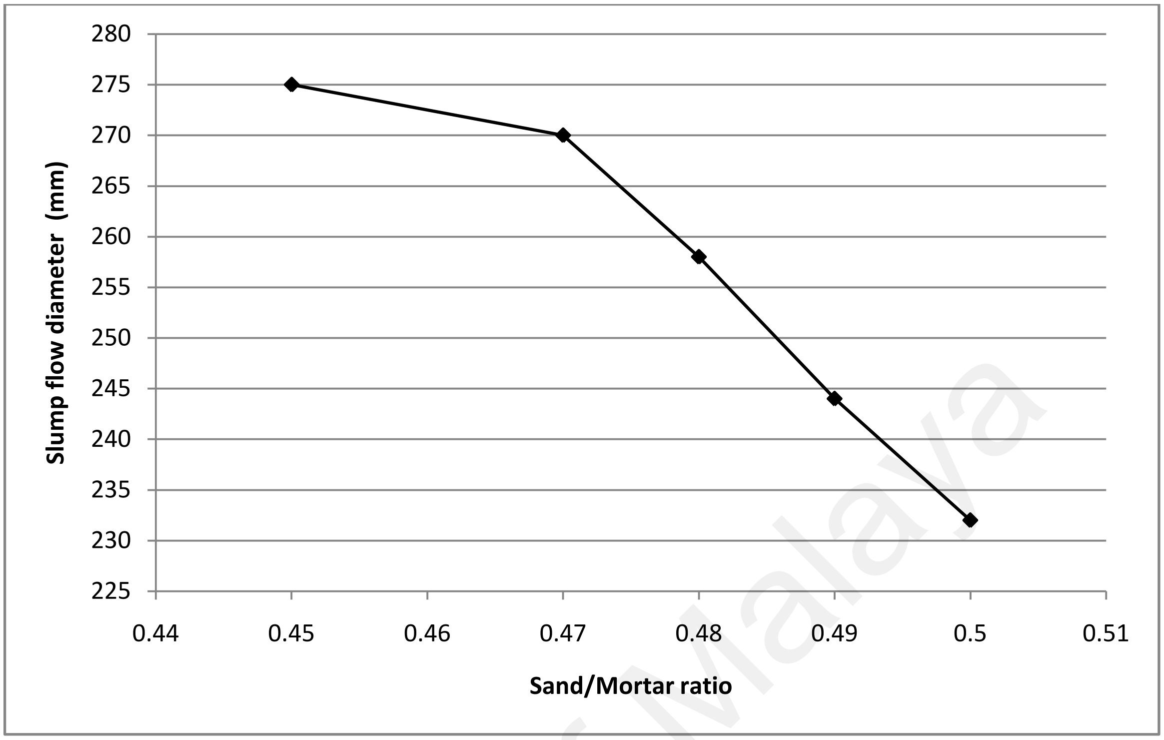


Figure 4.5: The relationship between the flow spread of mortar and S/M ratio

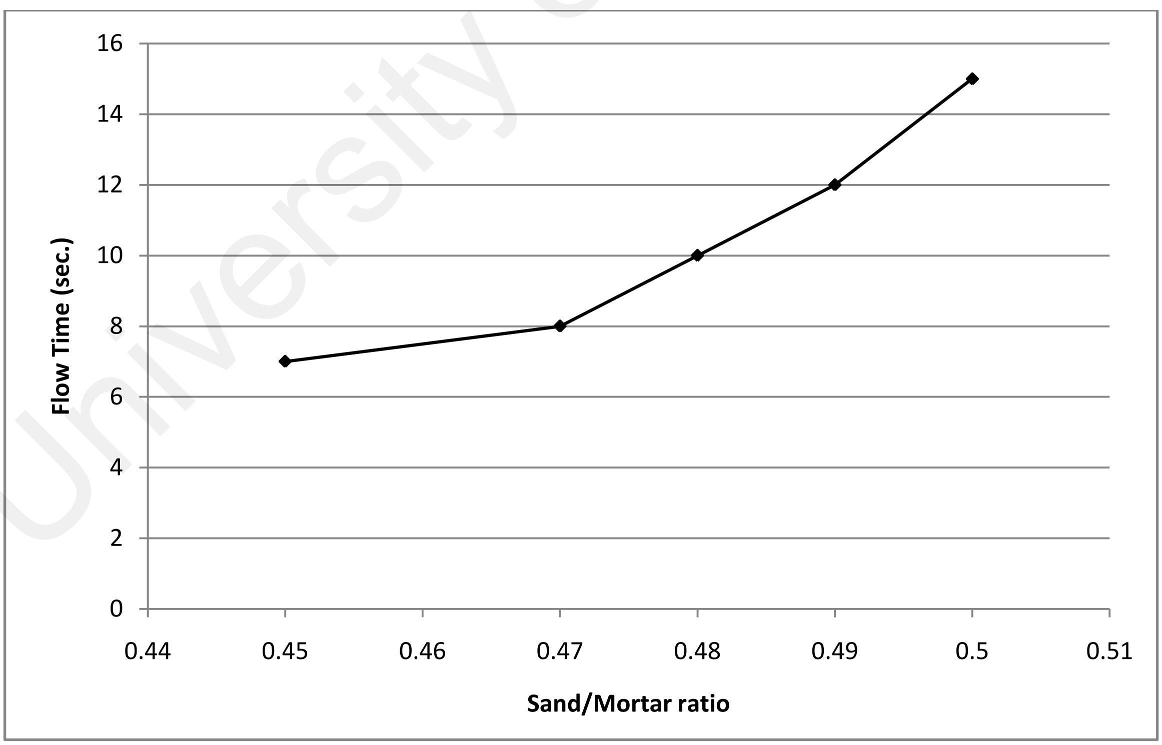


Figure 4.6: The relationship between the flow time of mortar and S/M ratio

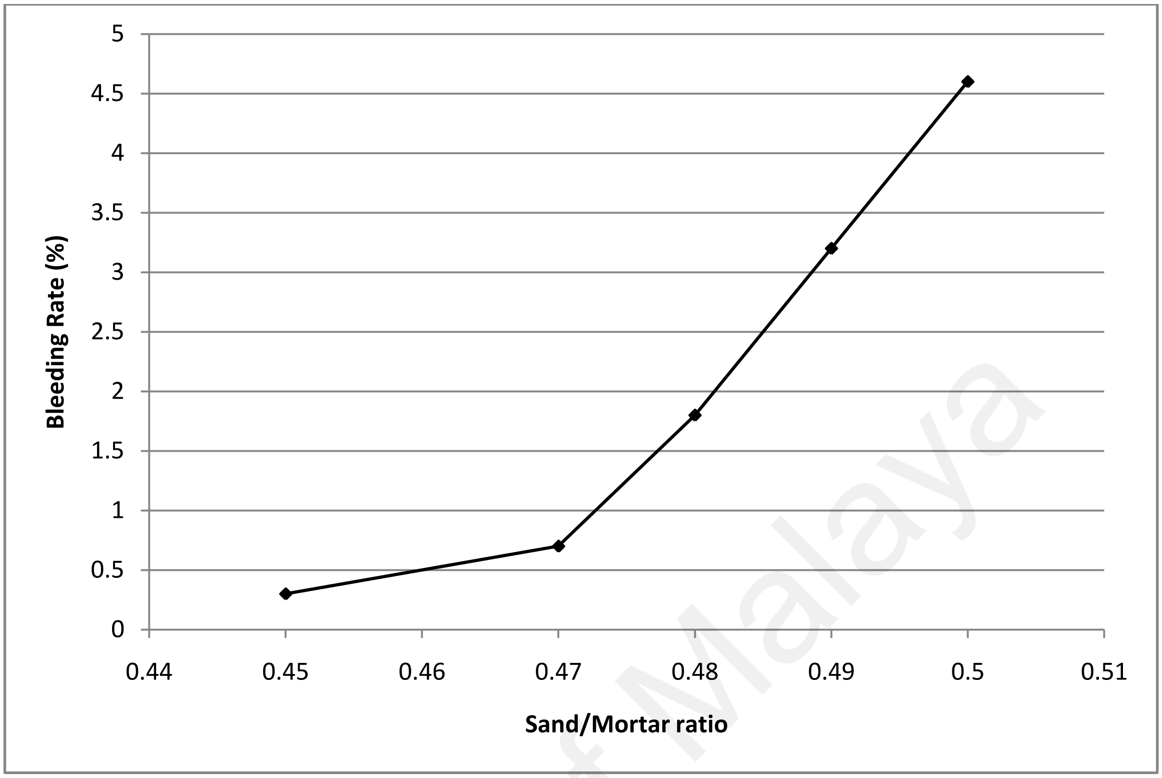


Figure 4.7: The relationship between the water bleeding rate of mortar and S/M ratio

4.4.2 Effect of fineness modulus of sand

Table 4.3 shows the effect of the fineness modulus of sand on flow ability and bleeding rate. The increase of fineness modulus leads to a decrease in the flow spread whilst increasing flow time. This could be due to the fact that coarse grains of sand need more cement paste to fill the voids between sand particles in order to reduce the bleeding of water. As is evident from Table 4.3, as the fineness modulus increases, the bleeding also increases which affects the filling ability of mortar.

Table 4.3: The different fineness modulus with flowability and bleeding rate

Fineness modulus	Slump flow dia. (mm)	Flow time (sec)	Bleeding rate (%)
2.50	270	8	0.7
2.90	234	12	4.2
3.10	210	17	6.7

4.4.3 Effect of pozzolan

The results of two types of the pozzolan used i.e fly ash and metakaolin are presented and discussed. Figure 4.8 shows the range of optimum dosage for the fly ash mortar mixes. It is apparent that increasing the fly ash replacement level decreases the optimum dosage. Depending on the replacement level, the optimum dosage ranges between 0.6 to 0.8% to give a slump flow spread in the range of 260 to 270mm. Figure 4.9 reveals that the range of optimum dosage for the metakaolin mortar mixes is between 1.6 to 2.5% to give a slump flow spread in the range of 260 to 280mm. It appears that as the replacement level of metakaolin increases the optimum dosage of superplasticizer also increases. Generally, the range of optimum dosage in metakaolin mortars is wider than the range of saturation dosage for the fly ash mixes. This can be attributed to the metakaolin particles being much finer and having an irregular shape. In general, the flow ability of fly ash and metakaolin mortars increase with higher superplasticiser dosages due to its dispersing and liquidising effect, making more free water available and thus giving a high flow spread in mortar mixes (Topçul and Uyguno, 2010). The superplasticizer dosage beyond the optimum dosage does not have any effect on the flow spread. In most cases it leads to the occurrence of segregation and bleeding on the surface and edges of the mortar spread as observed.

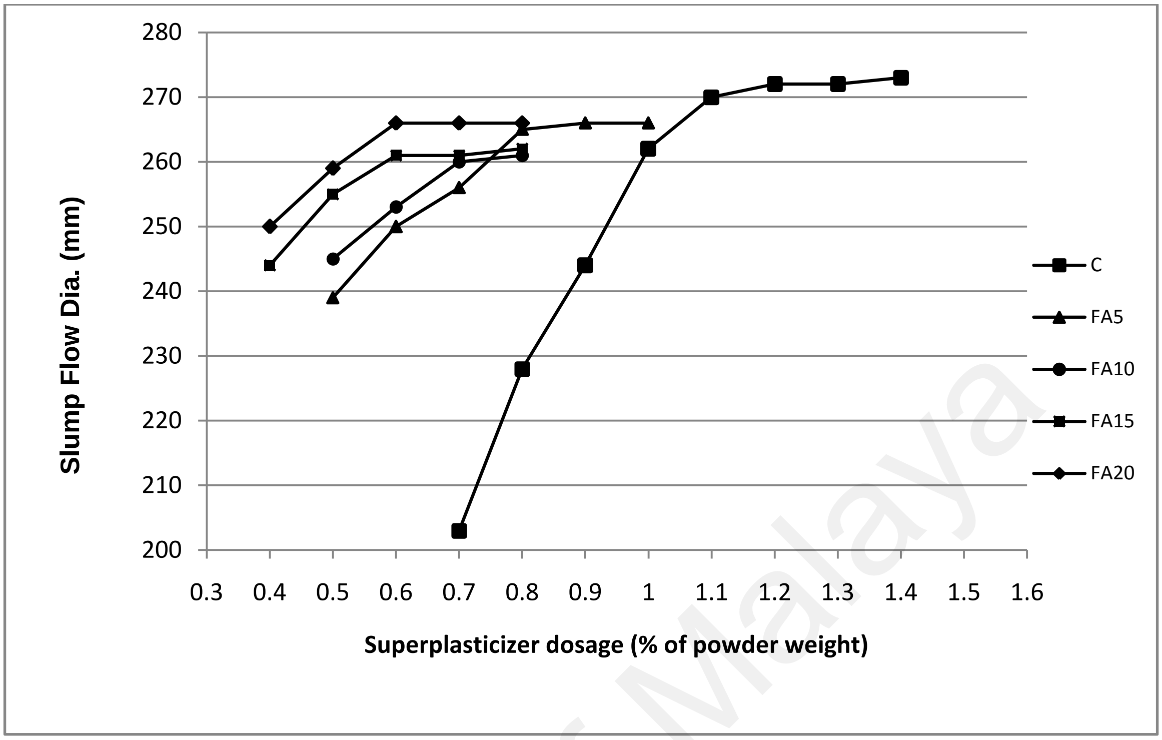


Figure 4.8: The optimum superplasticizer dosage of control and fly mixes

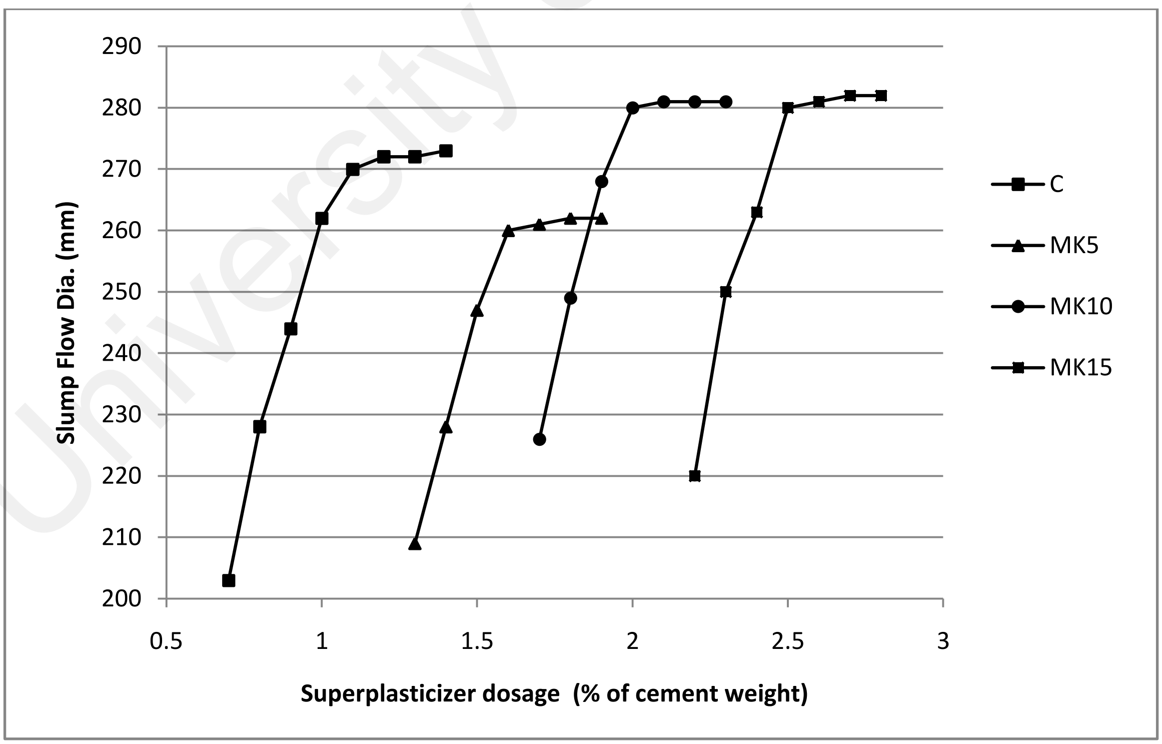


Figure 4.9: The optimum superplasticizer dosage of control and metakaolin mixes

The slump flow spread results of mortars of fly ash mixes ranged between 260 to 270mm, as shown in Figures 4.8. Figure 4.8 gives the superplasticizer dosage requirement for fly ash mortar mixes whereby increasing the cement replacement level led to a reduction in dosage to achieve the same workability. By increasing the replacement level, the volume of fly ash in the paste increases and since the specific gravity for fly ash is lower than cement this gives rise to lower mortar density. Moreover, fly ash has a spherical shape which easily rolls over one another (Topçul and Uyguno, 2010) and in turn reduces interparticle friction resulting in higher fluidity of the paste. Figure 4.10 highlights this effect for mixes with different replacement levels of fly ash but having the same water to powder ratio and superplasticizer dosage. The slump flow spread was higher.

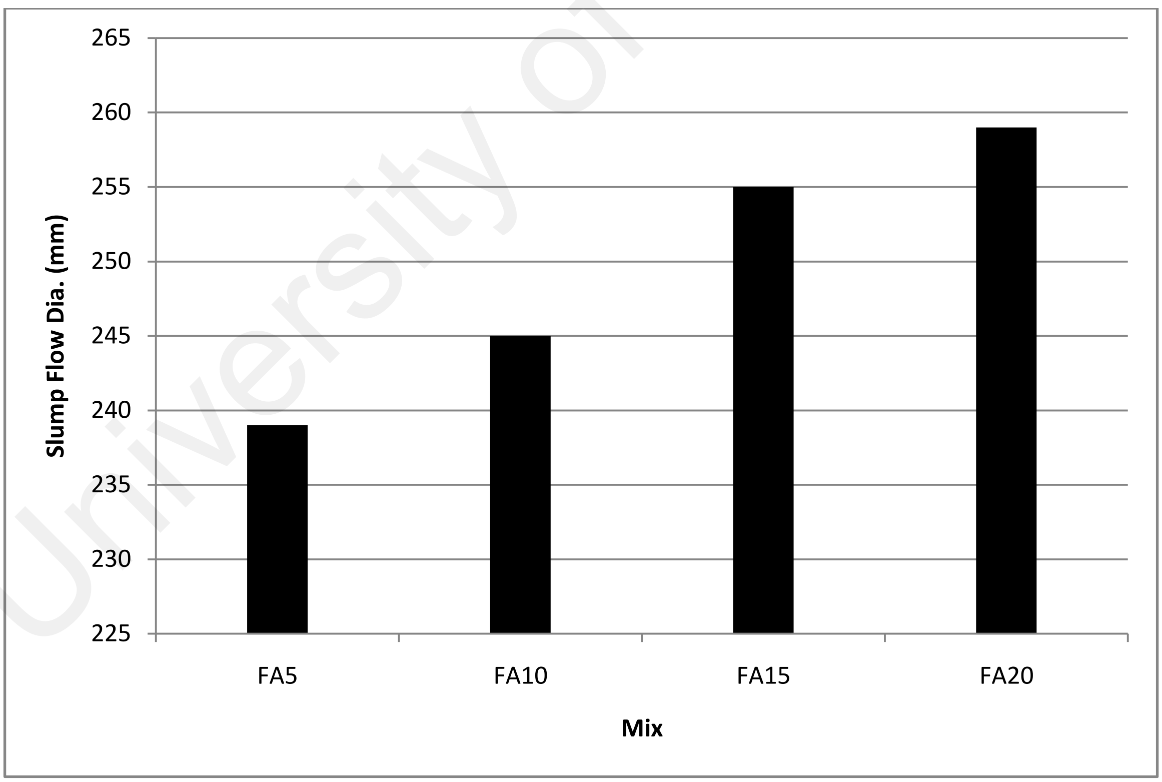


Figure 4.10: Flow spread of fly ash mixes with same w/p ratio and superplasticizer dosage

The slump flow of metakaolin mixes ranged between 260 to 280mm for the levels of replacement and as shown in Figure 4.9. Figure 4.11 illustrates the results of slump flow of metakaolin mixes at the same superplasticizer dosage. The results showed that an increase in the replacement level of metakaolin led to decrease in the flowability of mixes at the same w/p ratio and superplasticizer dosage. This can be attributed to the metakaolin particles being fine and possess an irregular shape. The irregular shape of the particles generates high frictional forces between them and consequently would lead to decrease in the flowability of the mixes.

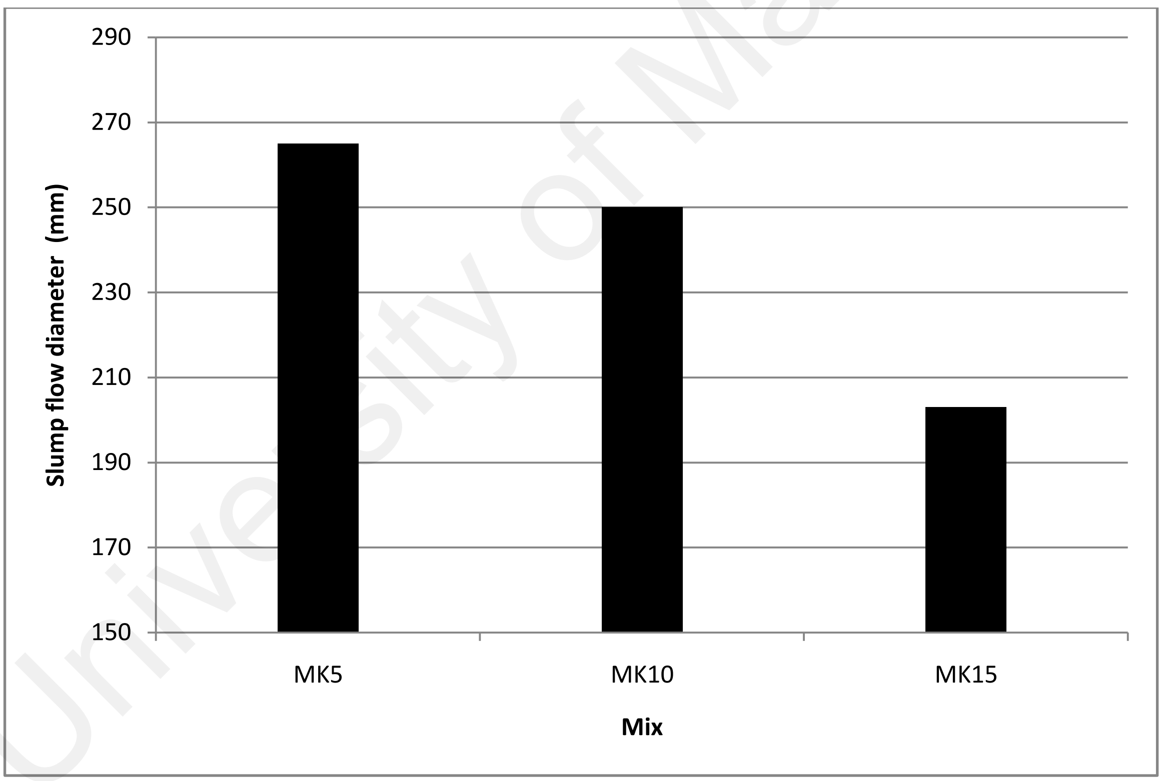


Figure 4.11: Slump flow diameter of metakaolin mixes with same w/p ratio and superplasticizer dosage

The results of the effect of the presence of fly ash in mortar on flow time from V-funnel test are presented in Figure 4.12. The results revealed that flow time decreases with an increase in replacement level of fly ash at the same dosage of supeplasticizer and w/p ratio. The flow time of the mini V-funnel box for fly ash mortars was lower as a result of decreasing viscosity of the paste and increasing paste content as the replacement level was increased.

The results of the effect of the presence of metakaolin in mortar on flow time are presented in Figure 4.13. Figure 4.13 showed that flow time of mini V-funnel increases with increasing replacement level of metakaolin at the same supeplasticizer dosage and w/p ratio. The increase of replacement level means increase in the paste content and the viscosity of paste due to increase interparticle friction. This different selection of superplasticizer dosage for the two pozzolan i.e fly ash and metakaolin is due to high difference between the ranges of superplasticizer dosages of such these pozzolan.

Figure 4.12: Flow time of fly ash mixes with same w/p ratio and superplasticizer dosage

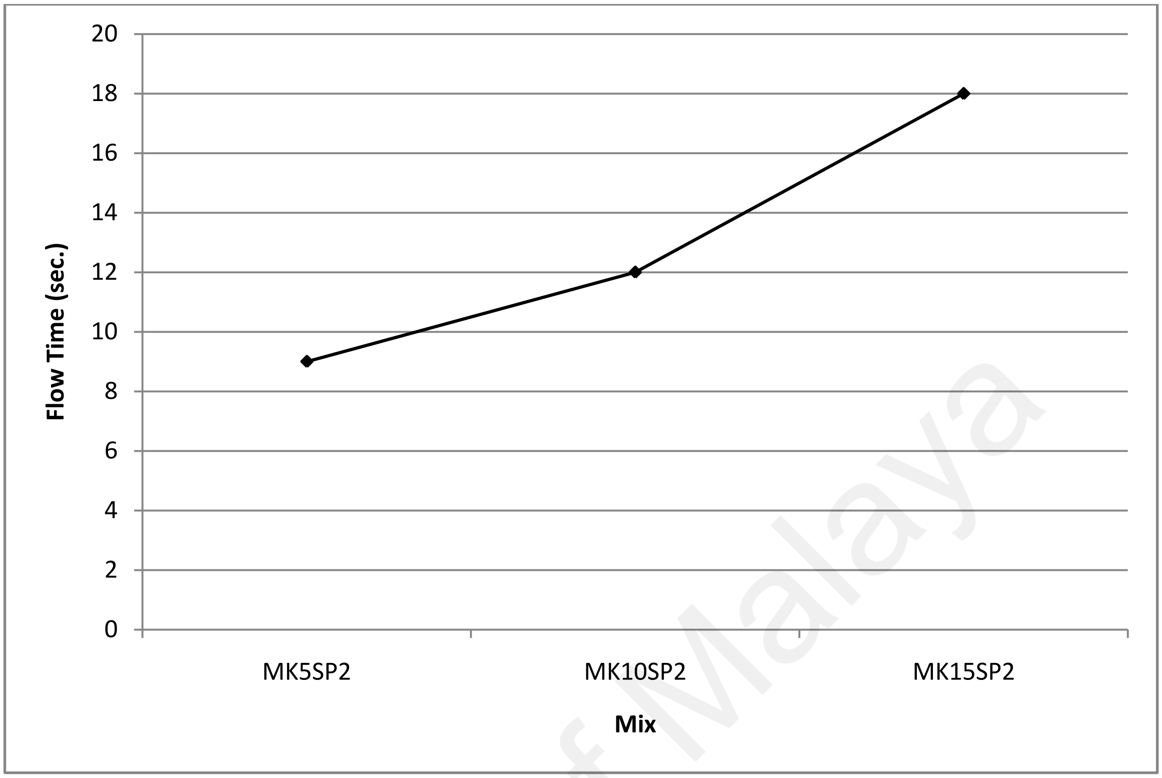


Figure 4.13: Flow time of metakaolin mixes with same w/p ratio and superplasticizer dosage

The setting time of concrete is defined as the initiation of solidification and hardening i.e. strength gain of concrete. The setting time results of fly ash and metakaolin mortar are shown in Figures 4.14 and 4.15 respectively. The results shown in Figure 4.14 indicated that the replacement of fly ash with cement retards the setting time. The reason for this retardation is the low cement content as compared with the control mix. The difference between the setting time results of fly ash mixes is slightly less compared to metakaolin mixes. This is because the converse effect of reduction of superplasticizer dosage with the reduction in cement content, which occurs with increasing replacement levels.

From Figure 4.15, the effect of both low cement content and high superplasticizer dosage on retardation of metakaolin mixes on setting time was observed. The final setting time of MK15 was too long due to the high dosage of superplasticizer and its retardation effect, and subsequently the test on MK20 was not conducted.

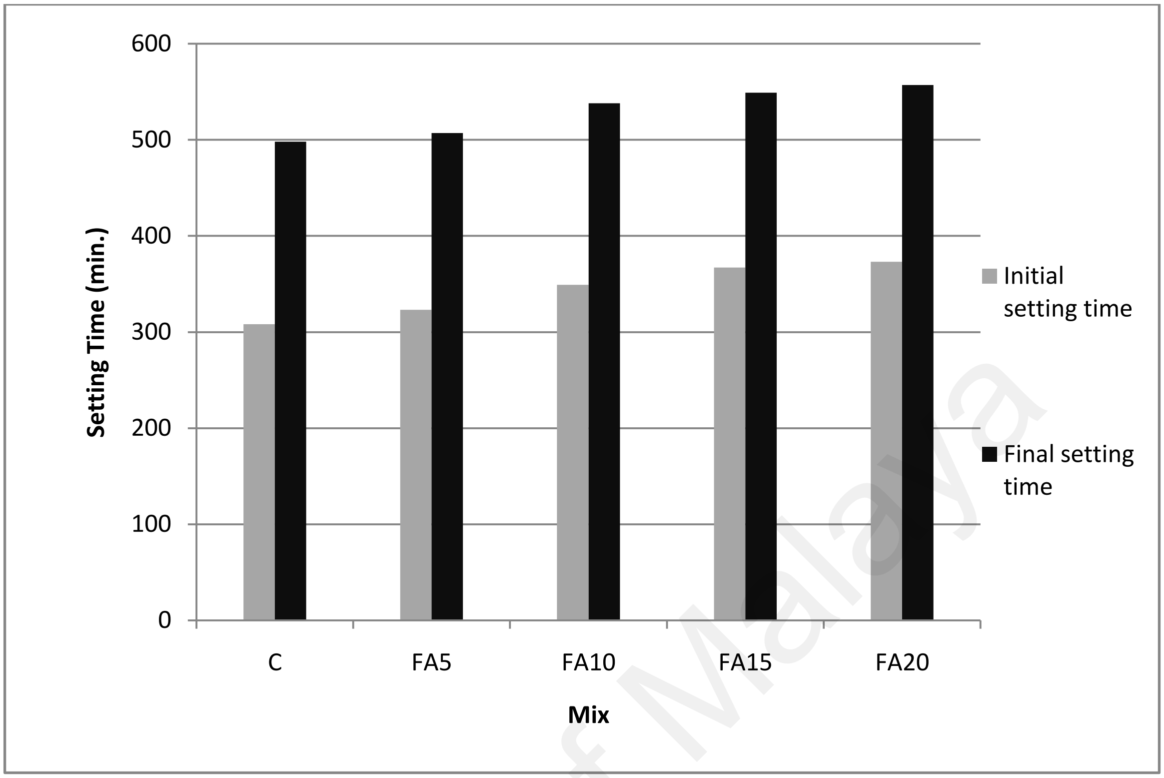


Figure 4.14: Initial and final setting time of control and fly ash mixes

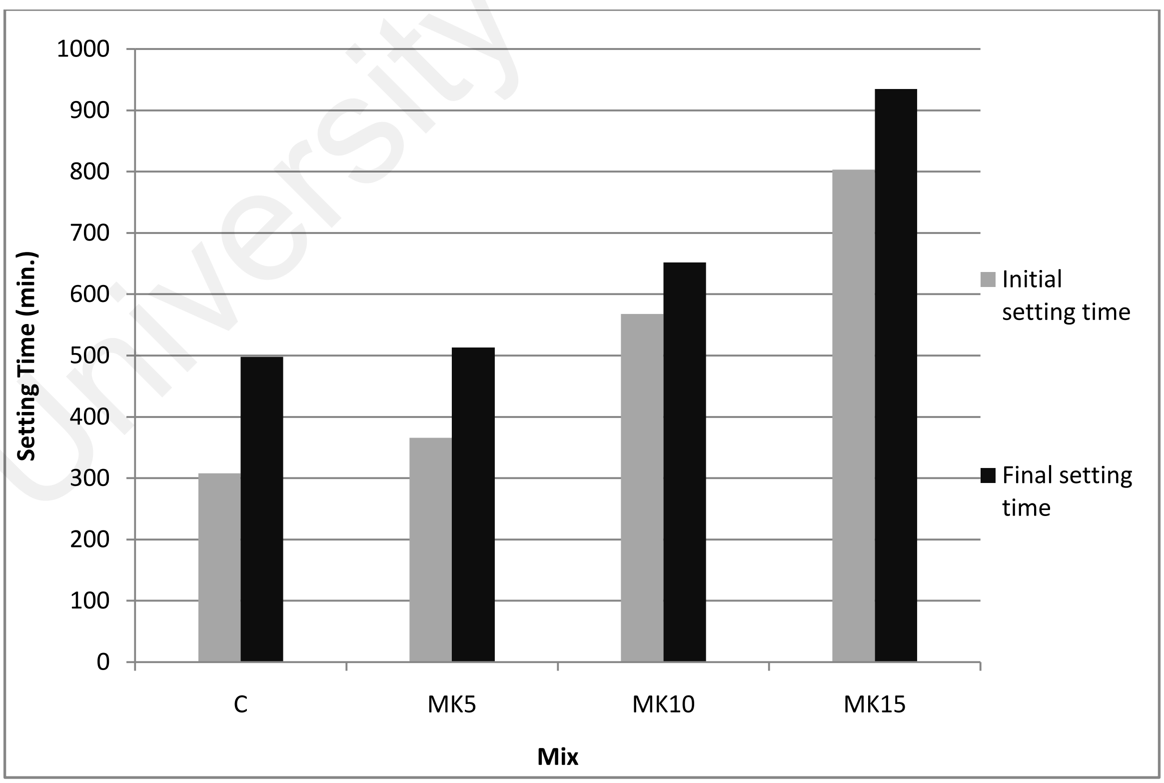


Figure 4.15: Initial and final setting time of control and metakaolin mixes

4.4.4 Effect of filler

The results of two types of the filler used i.e limestone powder and kaolin are discussed. The slump flow spread results of mortars of limestone powder mixes were between 195 to 275mm as shown in Figure 4.16. Figure 4.16 illustrates the range of optimum dosage for the limestone powder mortar mixes was between 1.2 to 1.6% depending on the replacement level. However, the increase in the replacement percentage led to increase in the superplasticizer dosage to achieve the same slump flow. However for the LP20 mix there was a decrease in superplasticizer requirement compared with the other limestone mixes. This may be attributed to the improved packing density of the powder and reduction in the voids at this percentage, thus decreasing the entrapped water in the mortar skeleton. Consequently, this will ensure adequate deformability of the fresh mortar, and agrees with the findings of Fujiwara et al. (1996).

The slump flow spread results of mortars of kaolin mixes were between 230 to 288mm, as shown in Figure 4.17. For the kaolin mortar mixes, increasing the kaolin replacement level increases the optimum dosage. The range for optimum dosage obtained was 1.4 to 3% to give a slump flow spread of 250 to 290mm depending on the replacement level and the range is wider compared to the limestone mixes. This can be attributed to the kaolin particles being much finer than the limestone particles.

Figure 4.18 illustrates the results of slump flow of limestone powder mixes at the same w/p ratio and superplasticizer dosage. However, the increase in the replacement percentage led to decrease in slump flow spread with the exception of LP20 which exhibited an increase in slump flow spread.

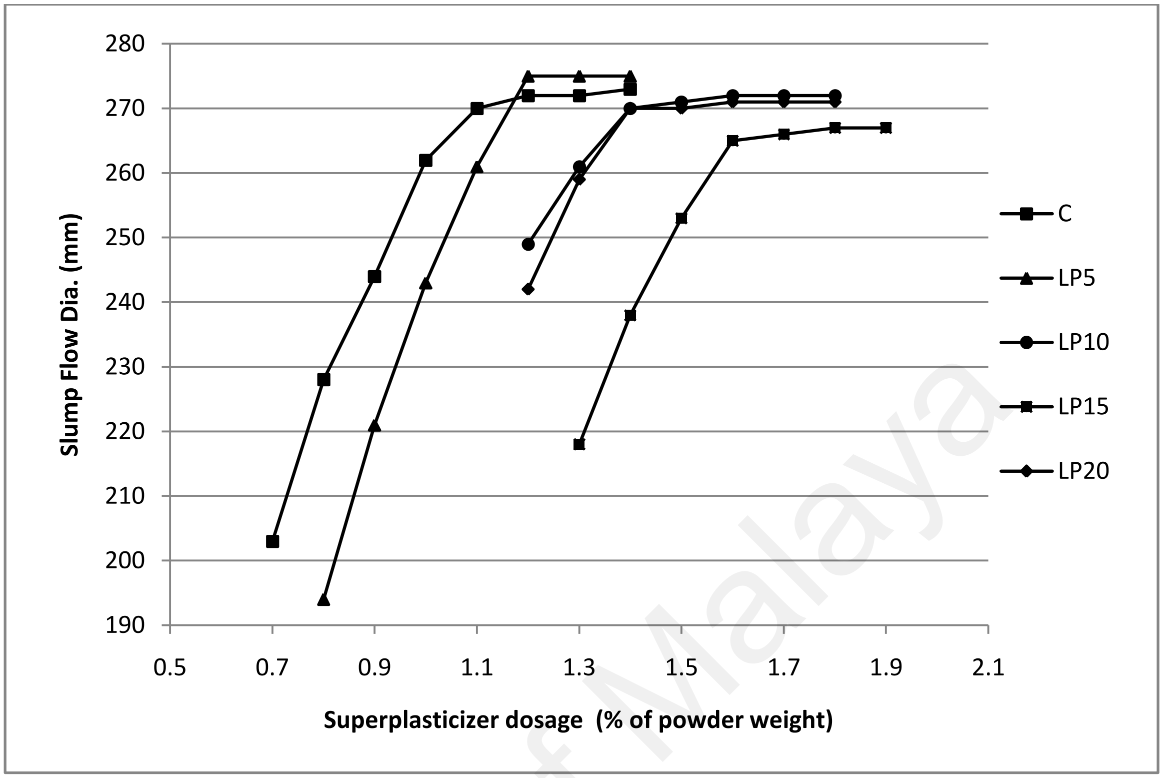


Figure 4.16: Optimum superplasticizer dosage of control and limestone mixes

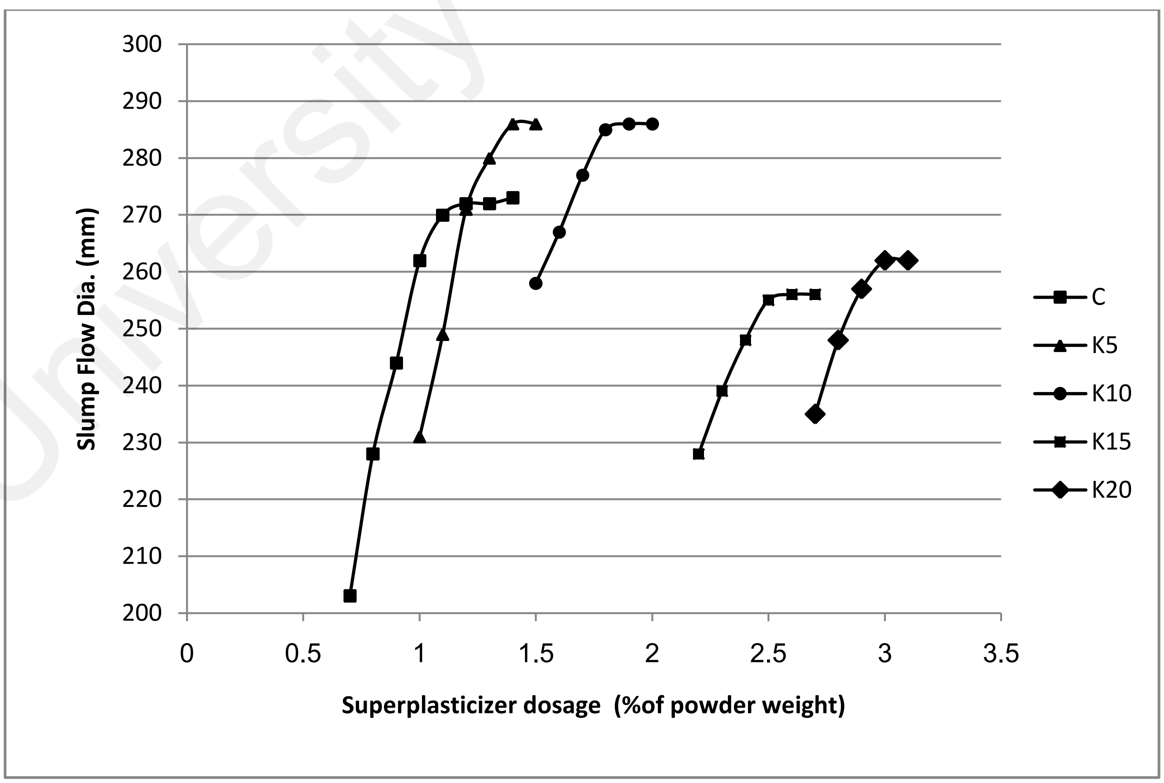


Figure 4.17: Optimum superplasticizer dosage of control and kaolin mixes

Figure 4.17 gives the superplasticizer dosage requirement for kaolin mortar mixes whereby increasing the sand replacement level led to an increase in dosage to achieve approximately the same workability. This is attributed to the finer and plate like structure of kaolin requiring more superplasticizer. However, using kaolin as a partial replacement of sand has enhanced the stability of self compacting mortar since it becomes more cohesive since the plate like structure leads to higher interparticle friction. Figure 4.19 shows the results of slump flow spread of kaolin mixes with w/p ratio of 0.32 and superplasticizer dosage of 2%. By increasing the replacement level there was a decrease in the flow spread due to increasing viscosity of the mortar mix. In addition, the superplasticizer dosage of 2% exceeded the optimum dosages for the K5 and K10 mixes, and this led to excessive bleeding for the K5 mix.

Figure 4.20 showed that the flow time for the V- funnel test increased with increase in the replacement level of limestone powder except LP20 for the same reason mentioned previously.

Figure 4.21 shows the flow time of kaolin mixes with the w/p ratio of 0.32 and superplasticizer dosage of 2%. By increasing the replacement level there was an increase in the flow time due to increasing yield stress and viscosity of the paste. In addition, the superplasticizer dosage of 2% exceeded the optimum dosages for the K5 and K10 mixes, and this led to excessive bleeding for the K5 mix. This different selection of superplasticizer dosage for the two fillers is due to high difference between the ranges of superplasticizer dosages of these fillers.

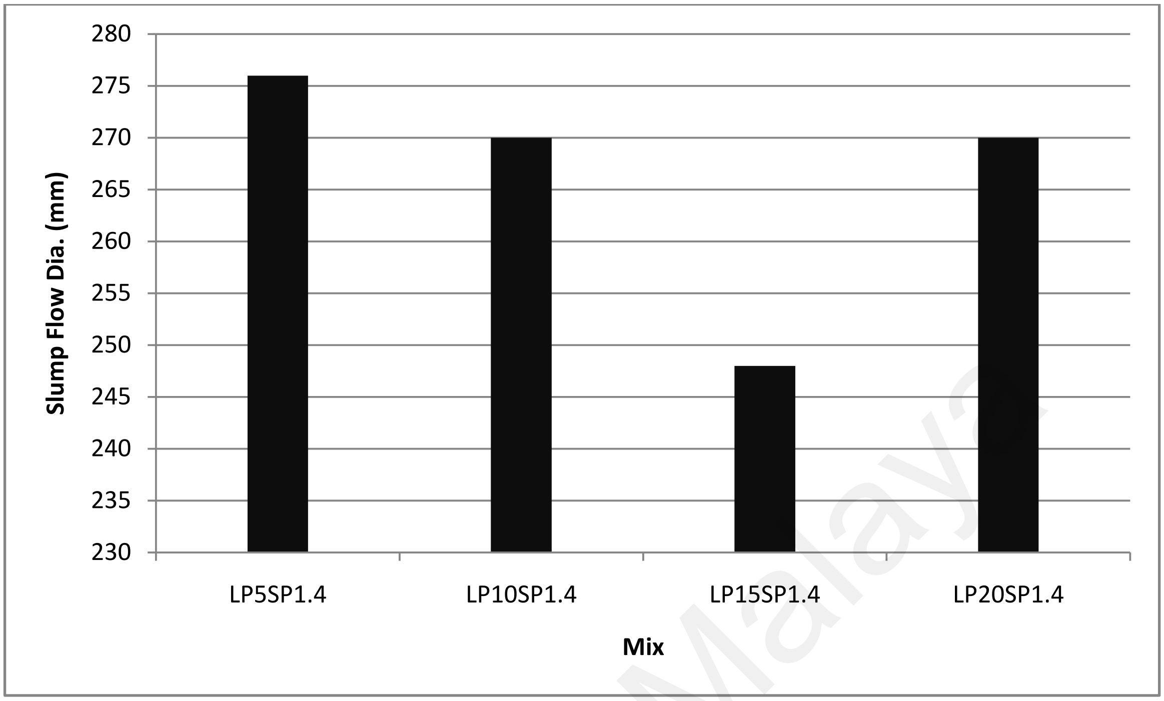


Figure 4.18: Flow spread of limestone mixes with same w/p ratio and superplasticizer dosage

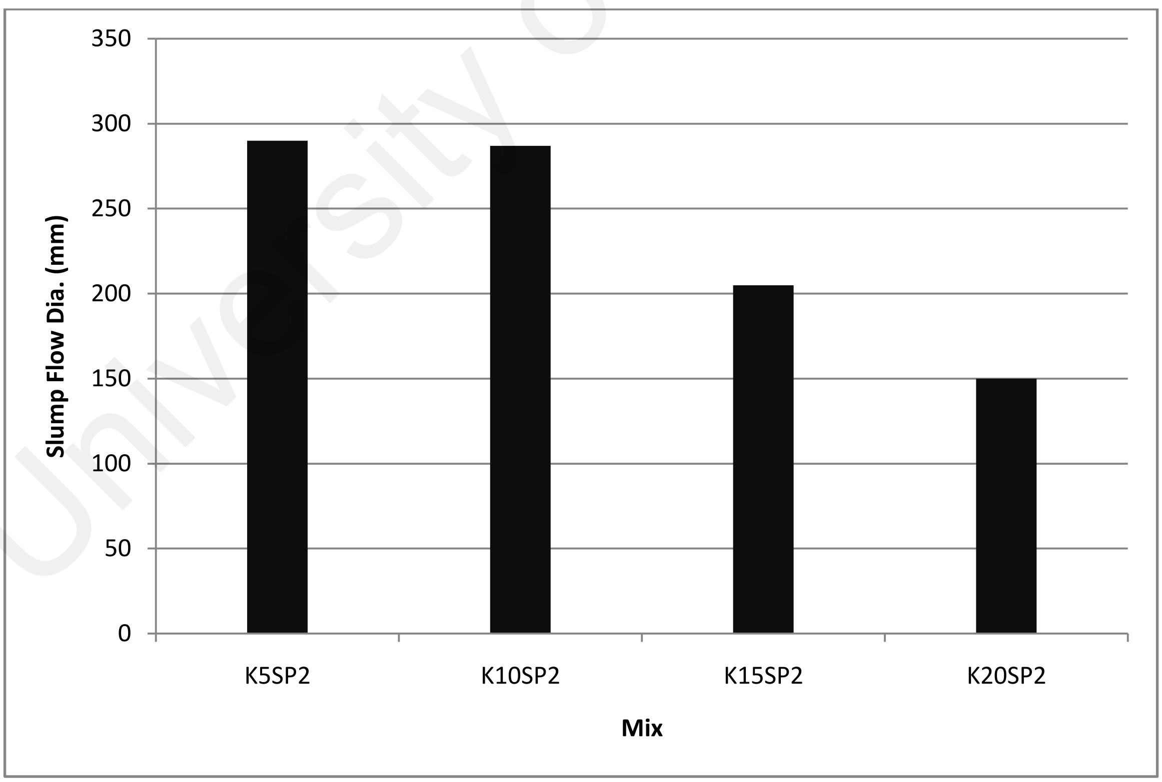


Figure 4.19: Flow spread of kaolin mixes with same w/p ratio and superplasticizer dosage

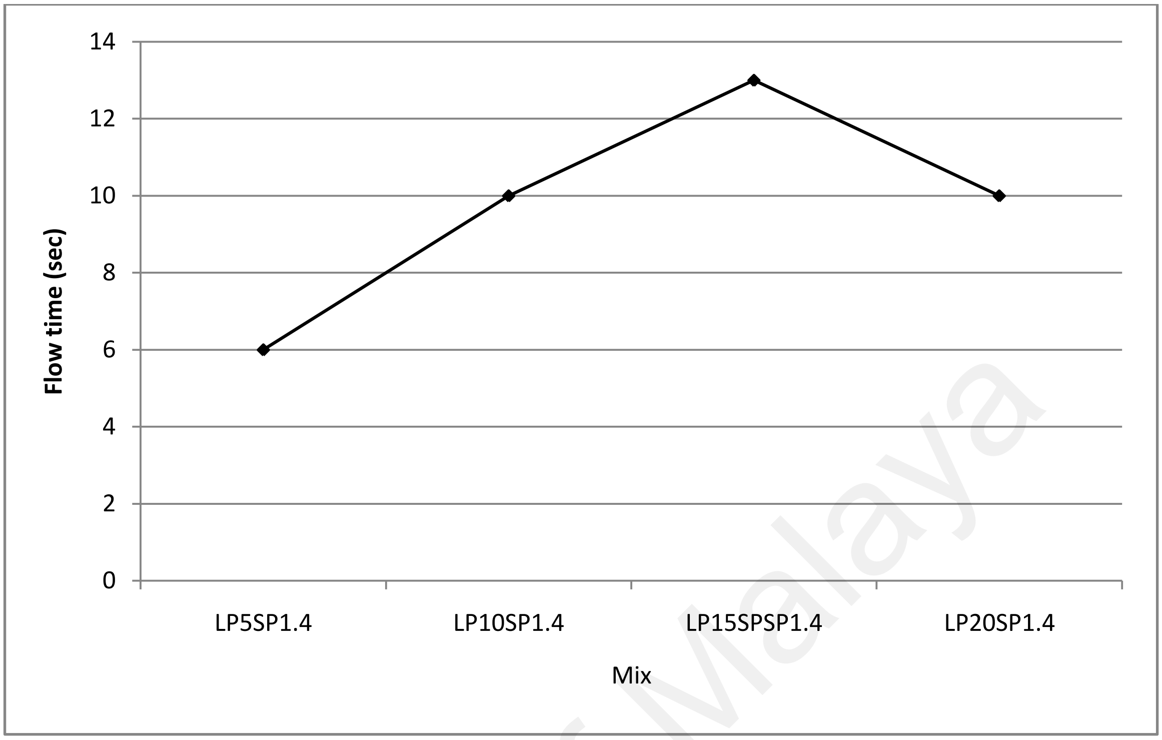


Figure 4.20: Flow time of limestone mixes with same w/p ratio and superplasticizer dosage

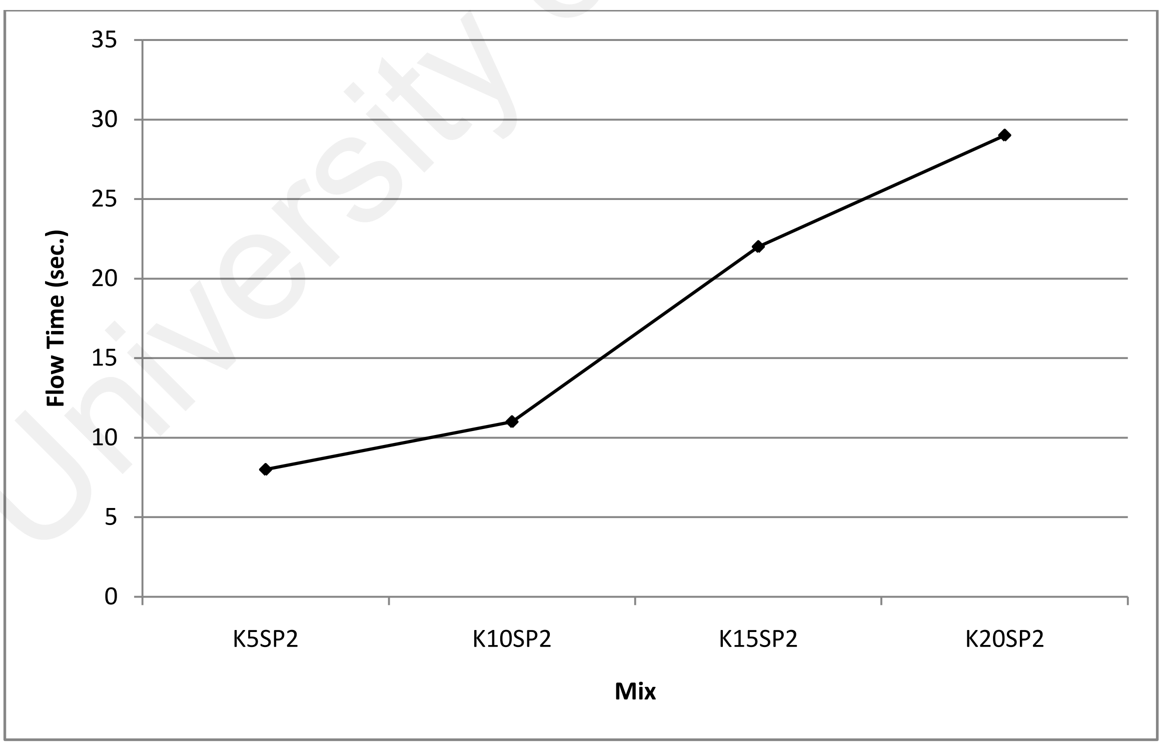


Figure 4.21: Flow time of kaolin mixes with same w/p ratio and superplasticizer dosage.

Generally, as observed from Figure 4.22, the effect of increasing the replacement levels of limestone powder with sand is to accelerate the setting time of mixes. This is the result of the effect of LP on the chemical reaction in the mixes, whereby addition of the $CaCO_3$ to Portland cement leads to the occurrence of two phenomena. Firstly, it accelerates the formation of ettringite during C_3A hydration. Secondly, it delays or stops the conversion process of ettringite to monosulfoaluminate depending on the amount of LP in the mixes. Thus, some of the sulfate ions can be interchanged leading to decrease in the setting time (Bonavetti et al., 2003).

Husson et al. (1992) and Barker and Cory (1991) mentioned that the addition of limestone to cement improved the formation of $Ca(OH)_2$ and C_3S hydration due to that limestone powder acts as nucleation sites and hence would decrease the setting time.

Ushiyama et al. (1986) reported that the addition of limestone with small amounts retards the early alite hydration, while large amount of limestone accelerates its hydration.

Figure 4.23 shows the setting time results of kaolin mixes where the results clearly revealed the effect of the replacement levels of kaolin on retardation of setting time. Where, as the replacement level is increased the retardation of setting time of kaolin mixes increased due to that this increase accompanied the increase in superplasticizer dosage. This retardation is considered large as compared with the limestone mixes because the effect of limestone on the hydration of cement as mentioned previously.

Christianto (2004) concluded that the increase in the replacement level of kaolin with cement led to retardation of setting time due to decrease in the amount of cement as well as the filler effect of kaolin.

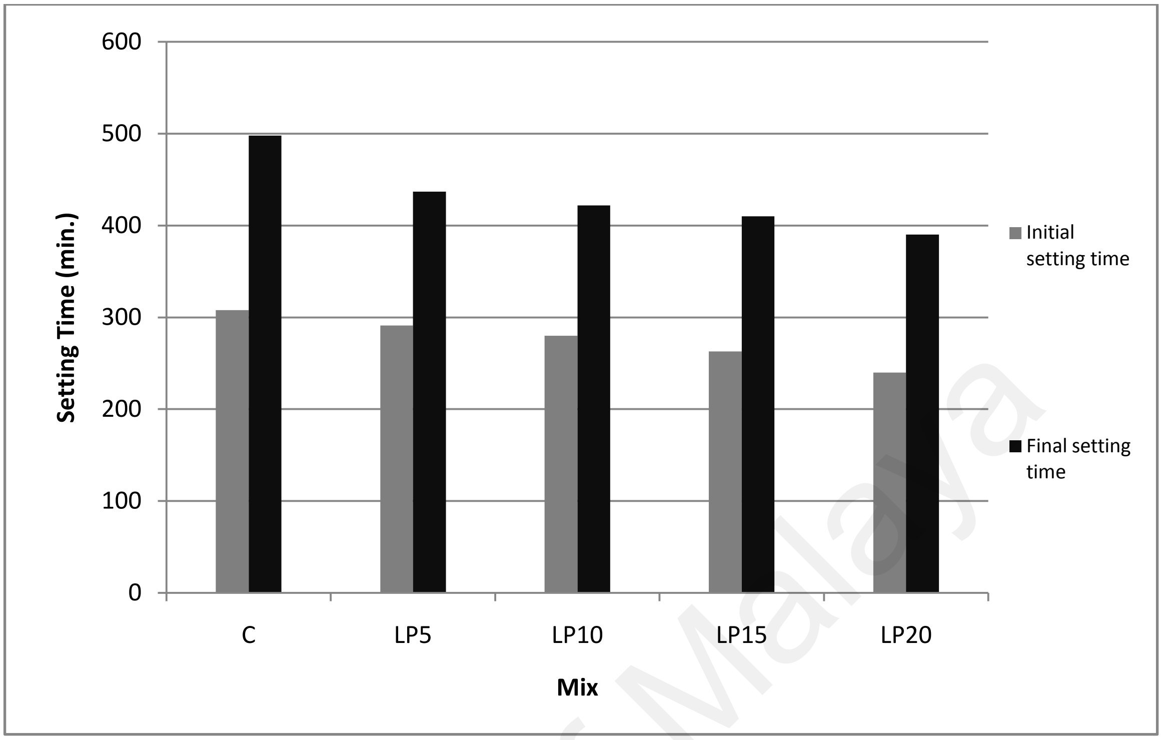


Figure 4.22: Initial and final setting time of LP mixes

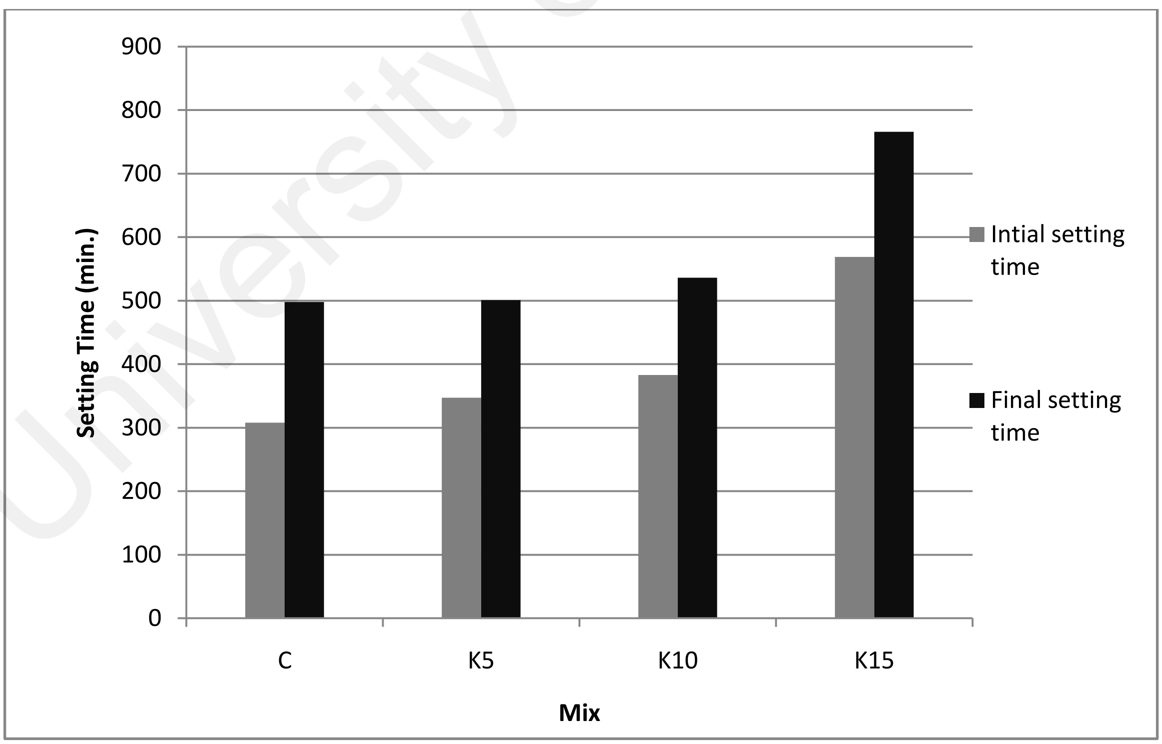


Figure 4.23: Initial and final setting time of K mixes

4.5 Hardened properties of mortar

The hardened test results of all mortar mixes are presented and discussed in this section. The effect of the dosage and type of pozzolan and filler on the hardened properties of mortar is discussed. Two kinds of hardened test i.e. compressive strength and water absorption tests were conducted.

4.5.1 Effect of pozzolan

Figure 4.24 illustrates the strength development of fly ash mixtures, where all fly ash mixtures show strength higher than the control mixture except for FA20 at 90 days. This is mainly due to the small amount of calcium hydroxide in the cement paste which reacts with fly ash to form calcium silicate gel, leading to an increase in compressive strength. In addition, high early age strength of the mixtures containing FA compared to the control mixture is due to the filler affect of fly ash. Fly ash particles are finer than particles of cement, and thus it would fill the gap between cement particles, and enhance the packing density of cement paste leading to increase in the strength. FA10 yields better results at all ages and seems to suggest that 10% replacement is the optimum. On the other hand, FA15 at early ages i.e. 3 days has strength lower than other fly ash mixes, but at 14 days and beyond, FA15 strength gives high strength when compared with FA5 and FA20. Thus FA10 is selected as the optimum replacement level for strength and as shown in Figure 4.25.

Metakaolin mixtures, namely MK5, MK10 and MK15 show good strength attainment compared with the control mixture due to the pozzolanic activity of metakaolin as shown in Figure 4.26. Figure 4.27 shows the relative strength of all metakaolin mixes, where it is apparent that the optimum replacement with metakaolin is 10%. The metakaolin which replaced cement has a significant effect on the Portland cement hydration since the Ca $(OH)_2$ which is produced during the cement hydration reacts with SiO_2 from metakaolin and leads to an augmentation in the interfacial transition zone densification. This improvement in the transition zone density leads to a better bonding strength in this zone and thus the microcracking is decreased. Therefore, the initiation of microcracking takes place at a high stress level and this is in agreement with the findings of Aulia and Deutschmann (1999).

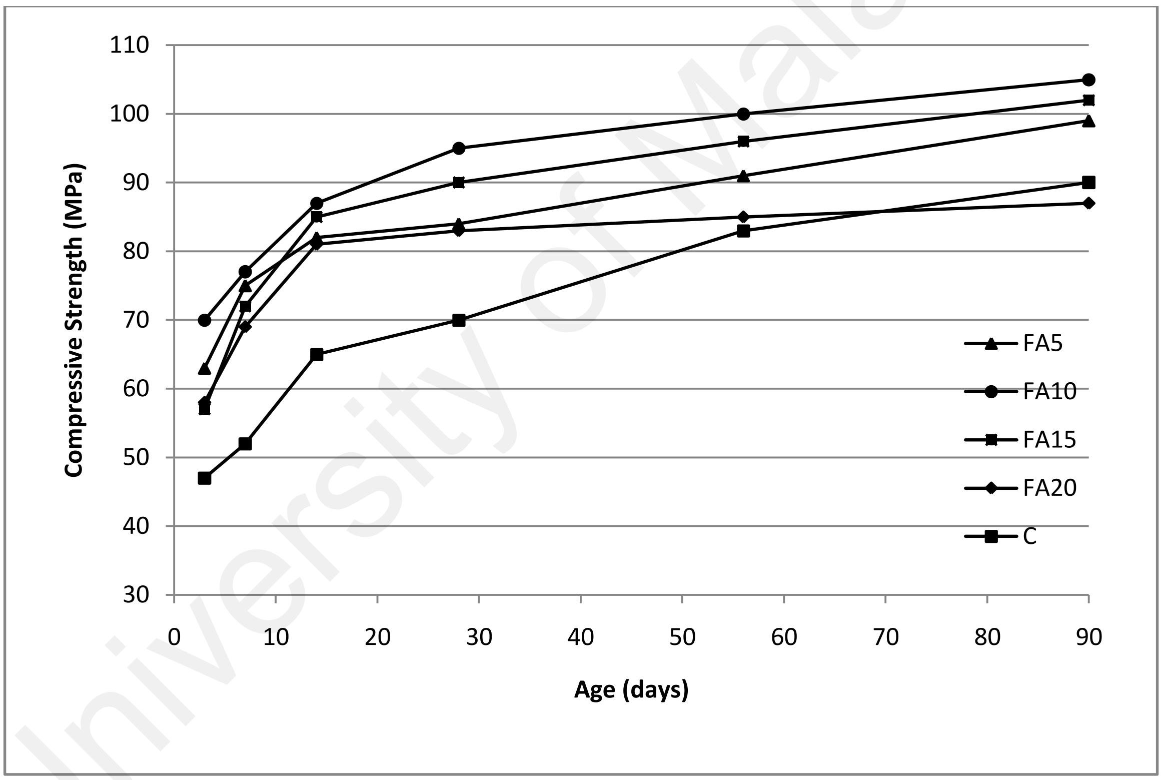


Figure 4.24: Compressive strength of fly ash mixes with age

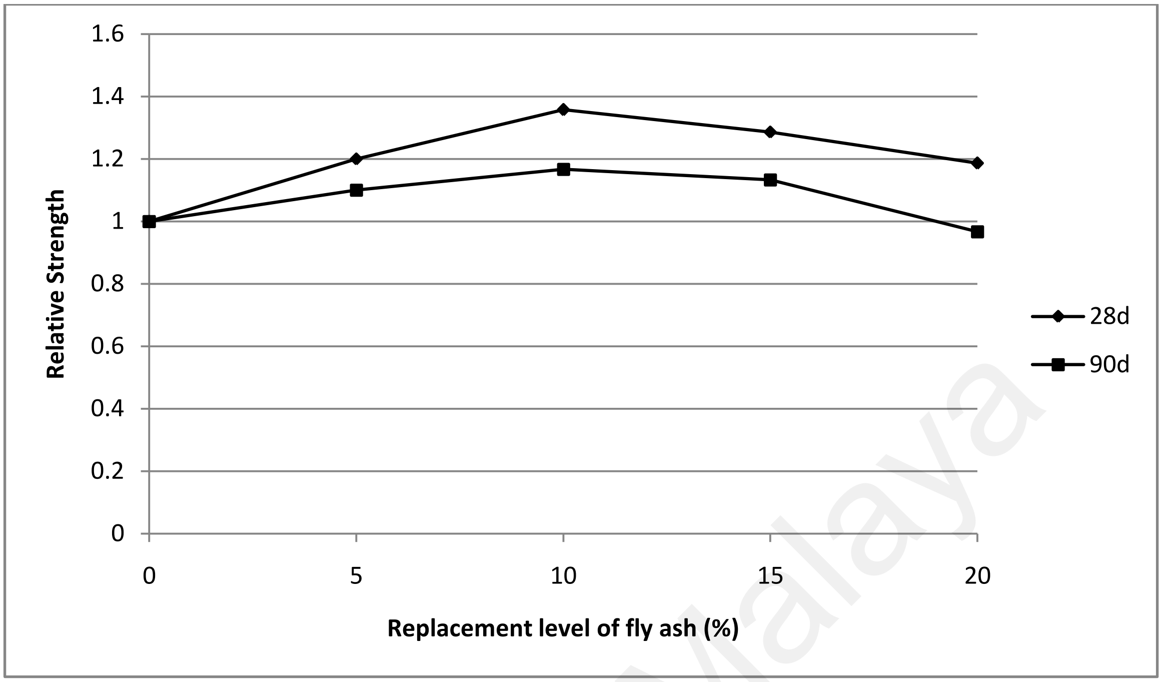


Figure 4.25: Relative strength of fly ash mixes at 28 and 90 days

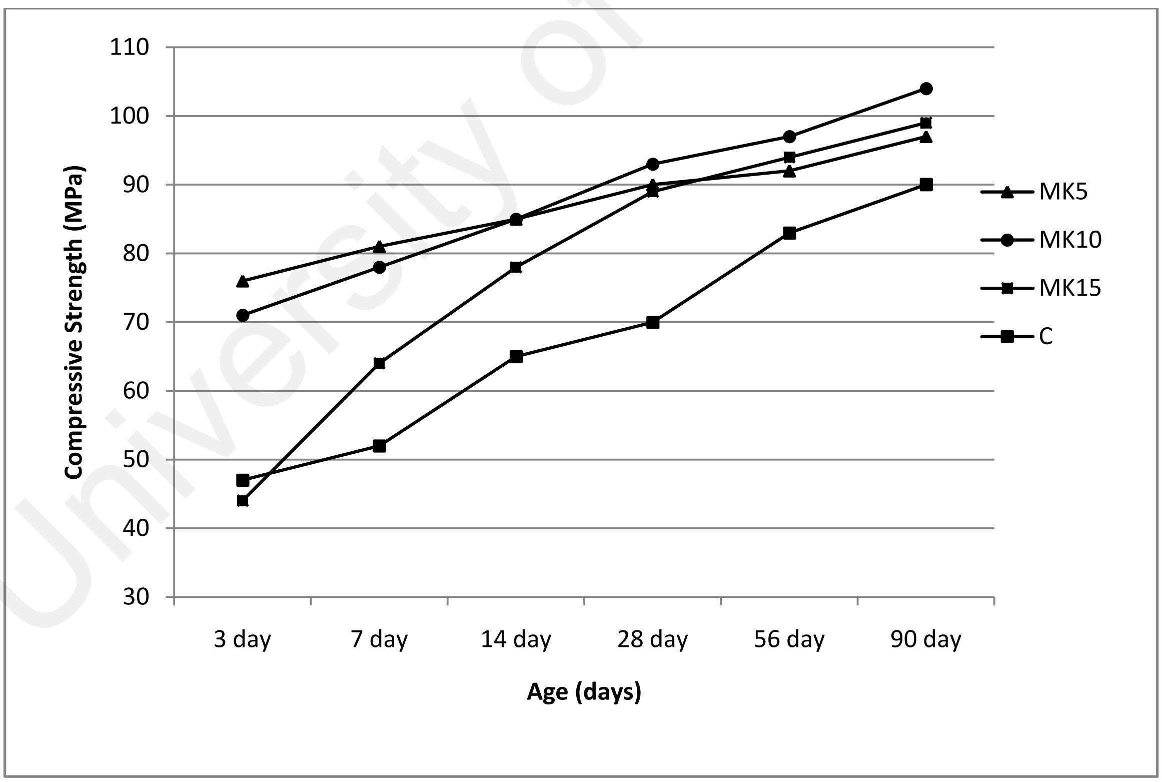


Figure 4.26: Compressive strength of metakaolin mixes with age

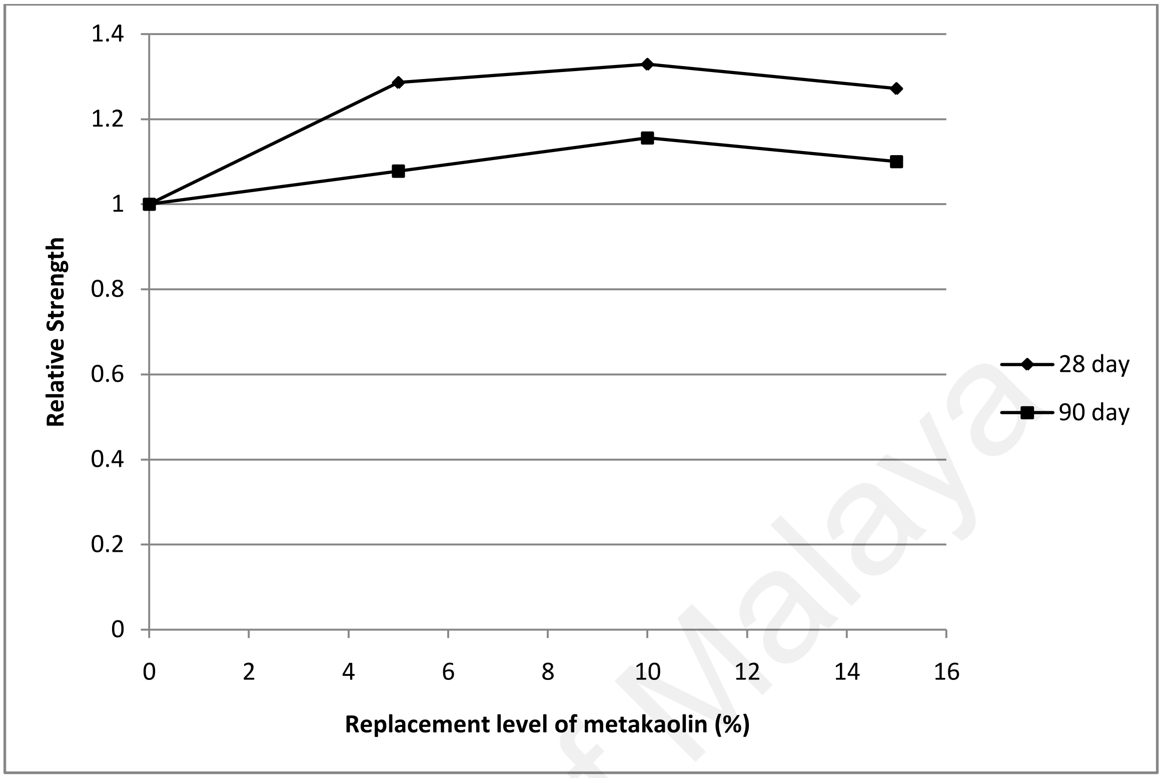


Figure 4.27: Relative strength of metakaolin mixes at 28 and 90 days

The effect of the employment of pozzolanic materials on water absorption of mortar at 28 days is presented in Figures 4.28 and 4.29. Figure 4.28 illustrates the effect of FA replacement level with cement on water absorption of the mixes. Water absorption of FA5 is higher than water absorption of the control mix, while water absorption of FA10 gave lower water absorption value as compared with control mixture. When the levels of fly ash increased after 10% replacement of cement weight the water absorption increased. This may be attributed to fly ash acting as a dilator of pores between cement particles. As deduced by Liu (2009). In addition to that, specific gravity of fly ash is low and this would increase the paste volume and thus would increase water absorption of mixes (Dinakar et al., 2008). The low water absorption value of FA10 mix resulted from the low porosity of the mix due to the pozzolanic activity of fly ash as compared with the control mix. The pozzolanic activity plays a significant role in reducing the calcium hydroxide and pore refinement (Chindaprasirt and Rukzon, 2008).

Figure 4.29 indicates that as the replacement level of metakaolin with cement increased the water absorption decreased until the 10% replacement level. At 15% replacement level, the water absorption of the mix is more than the water absorption of MK10 and less than water absorption of MK5. The reduction in water absorption of MK10 could be due to the combined effect of the pore refinement obstructing flow of water and the dense packing of the paste structure which in turn would reduce the porosity. The pore refinement of paste structure leads to stop flow of water in these mixtures (Gonçalves et al., 2009).

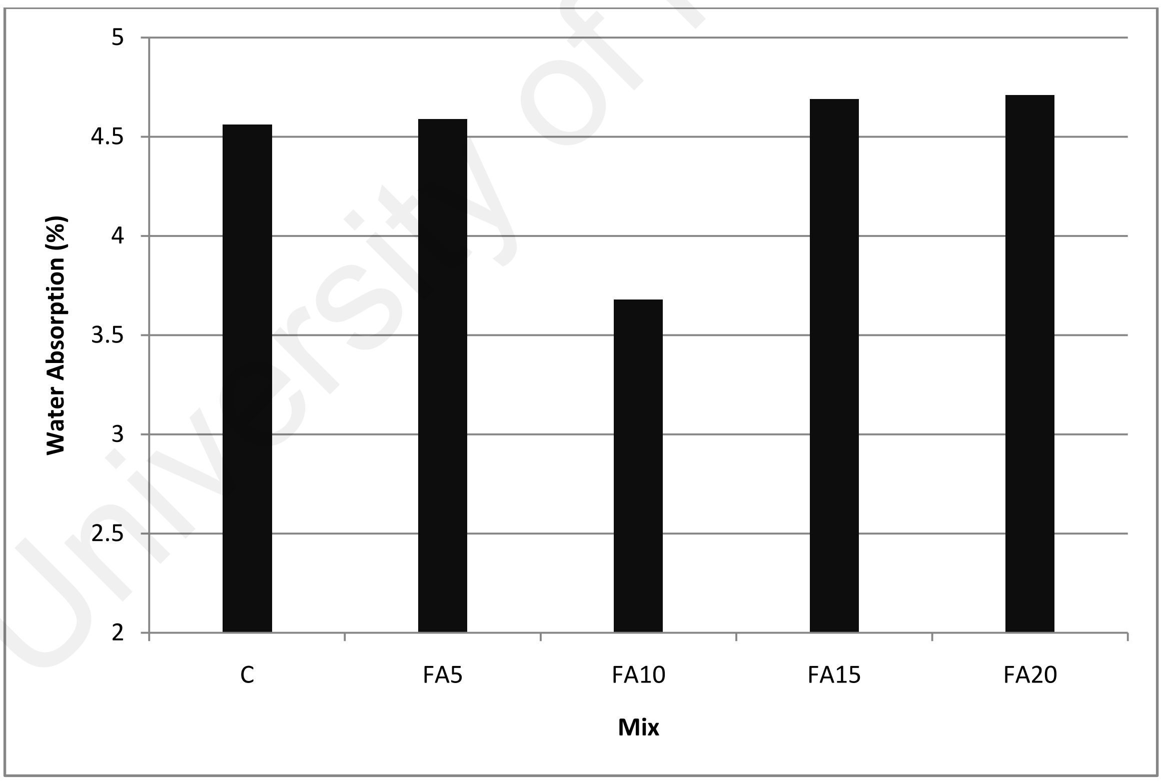


Figure 4.28: Water absorption of all fly ash mixes and control mix

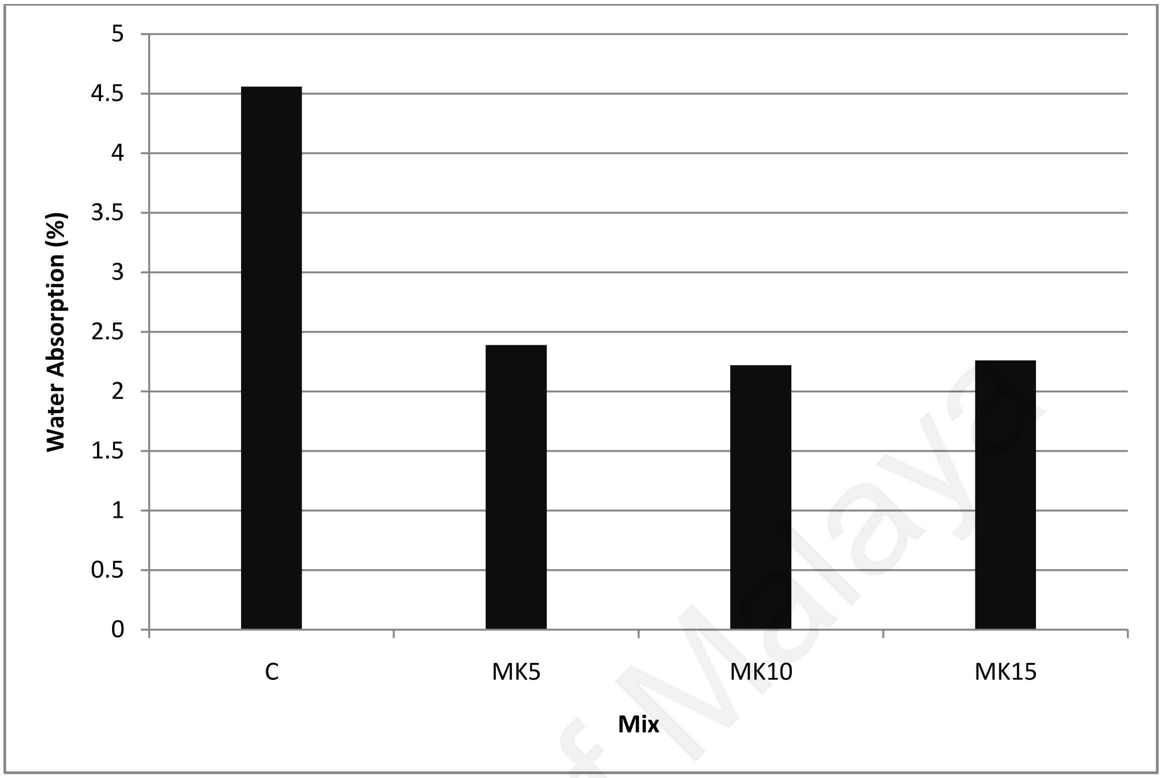


Figure 4.29: Water absorption of all metakaolin mixes and control mix

4.5.2 Effect of filler

It is apparent that the mixtures for all the limestone powder (LP) replacement percentages for sand resulted in higher strength when compared to control mixture C as shown in Figure 4.30. As the LP replacement level is increased, the strength also increased. The increase of strength for LP mixes is due to two reasons. Firstly, the limestone powder acts as filler i.e. physical effect because its fineness is higher than that of cement. Hence, it fills voids between cement particles and increases the packing density and thus yields high strength and good durability. Secondly, the chemical effect of limestone powder on the hydration reaction of cement compounds i.e. C_3S and C_3A which is indirect (Bustnes, et al., 2004). Ramachchandran (2001) concluded that hydration of C_3S will accelerate following the addition of more LP. It can be said that the limestone has a physico-chemical effect. After 28 days of curing, the relative strength of LP5, LP10, LP15 and LP20 when compared to C are 1.3, 1.33, 1.47 and 1.57 as shown in Figure 4.31. The optimum mix for the LP mixtures is LP20 for higher strength.

From Figure 4.32, it can be observed that adding kaolin as a replacement of sand leads to a decrease in the strength when compared with the control mixture. After 28 days, the relative strength of K5 is 0.94, while after 90 days the relative strength is reduced to 0.98 as shown in Figure 4.33. Although silica content in kaolin is high, the strength of kaolin mixes is low. This could be because, in the case of kaolinite, the octahedral layers, which are centered on aluminum cation, have hydroxyl groups (see 2.4.2.3) on their external surfaces. These most probably do not interact strongly with the hydroxyl ions in cement (Konan et al., 2009). In addition to this, Figure 4.33 indicates that the increase in the dosage of kaolin leads to reduction in strength at 90 days. However, the difference in strength of the K5 and control mixture decreases with the increase in age until they are almost the same at 90 days as shown in Figure 4.32.

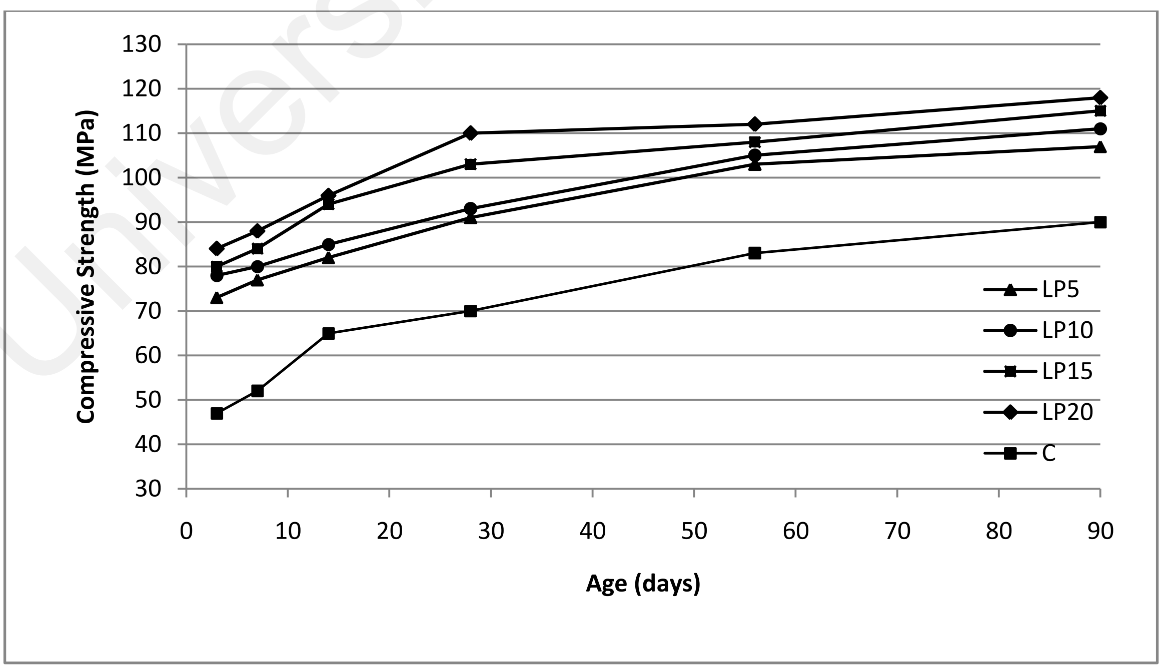


Figure 4.30: Compressive strength of limestone mixes with age

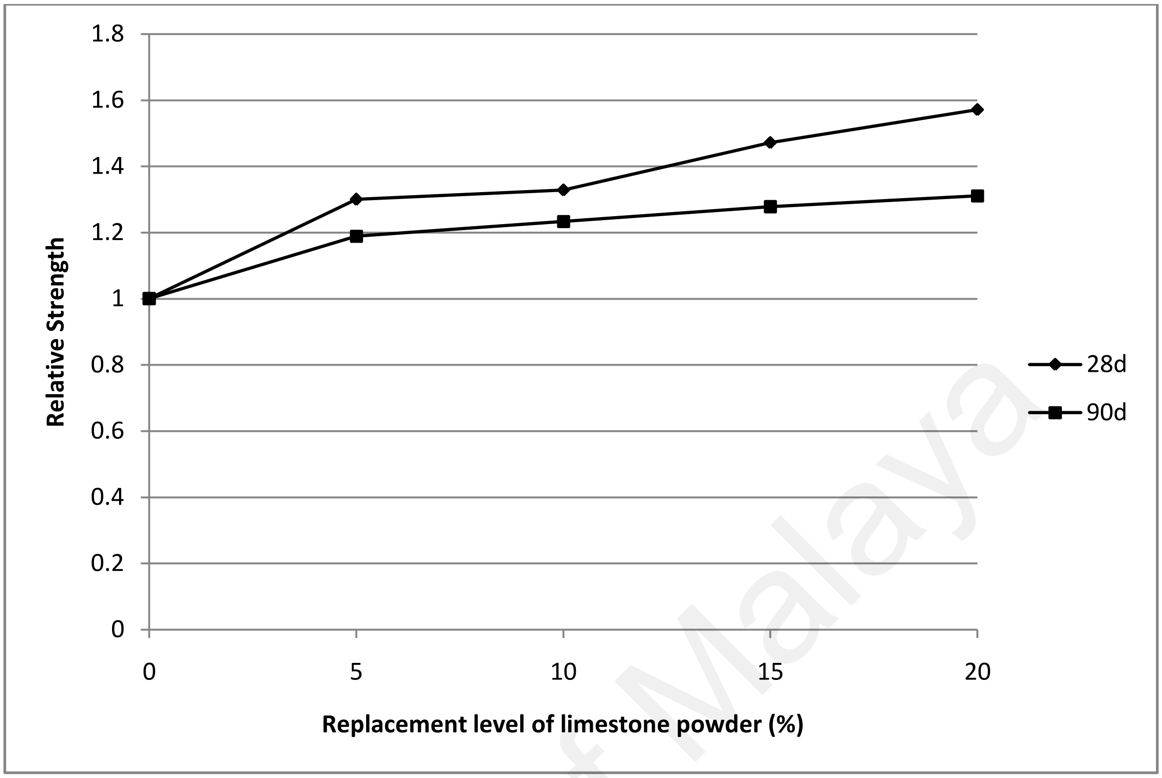


Figure 4.31: Relative strength of limestone mixes at 28 and 90 days

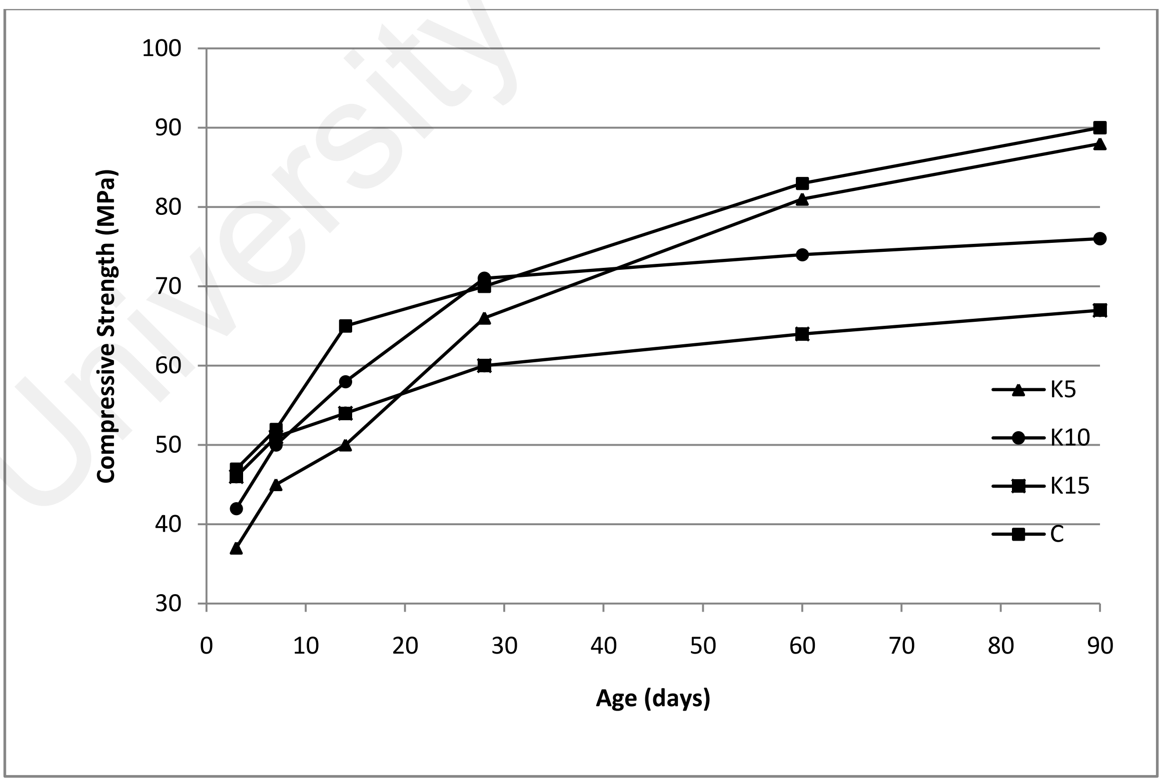


Figure 4.32: Compressive strength of kaolin mixes with age

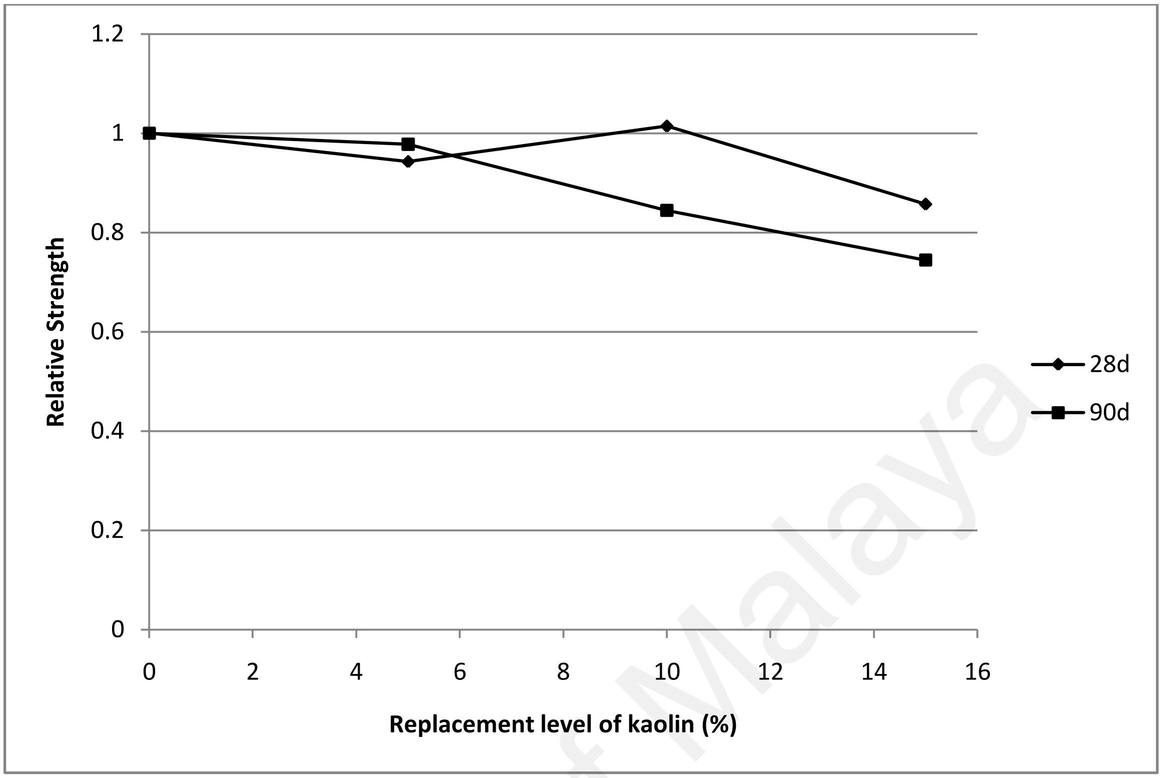


Figure 4.33: Relative strength of kaolin mixes at 28 and 90 days

The effect of the use of these filler materials on water absorption of mortar at 28 days is presented in Figures 4.34 and 4.35. From Figure 4.34, the results revealed that LP mixes have low water absorption as compared with the control mix. In addition to that, as the replacement level of LP increased the water absorption of mortar decreased. This agrees with the results of the study conducted by Guemmadi et al. (2009). The increase in replacement level of LP led to better packing density of the mixes and thus reduced the water absorption. However, Figure 4.35 showed that the employment of kaolin as replacement for sand gave higher water absorption values than the control mix.

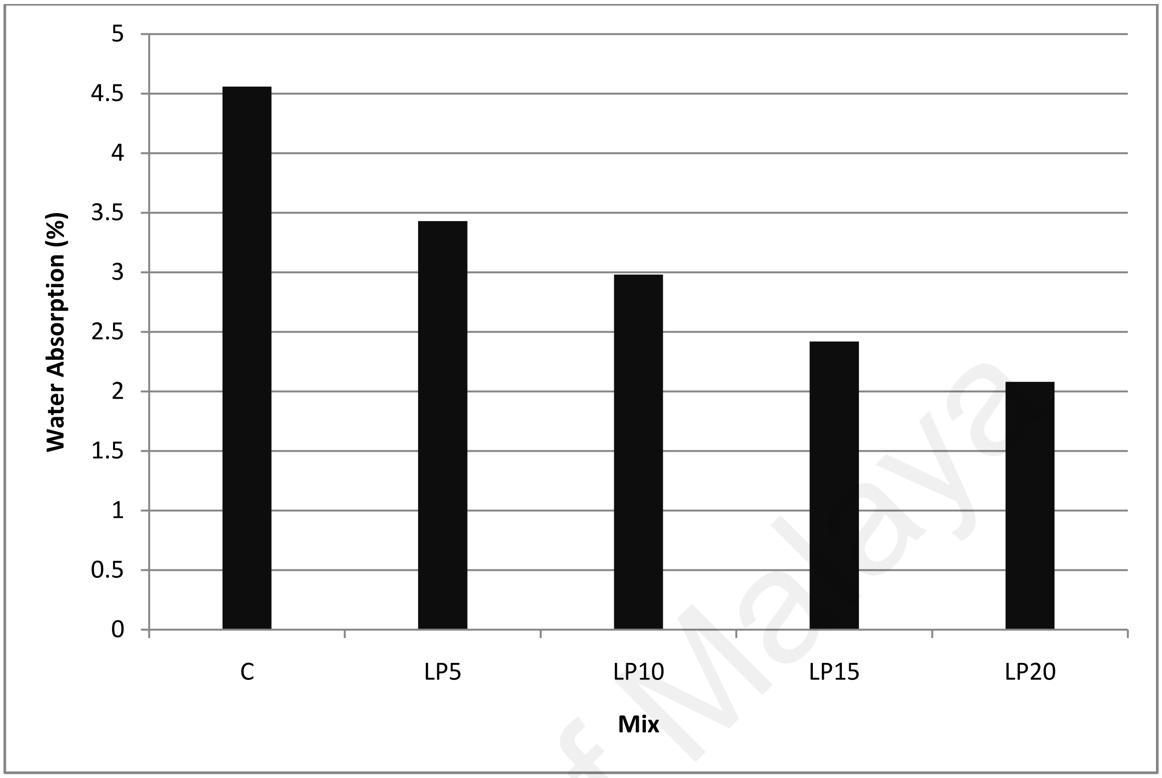


Figure 4.34: Water absorption of all limestone mixes and control mix

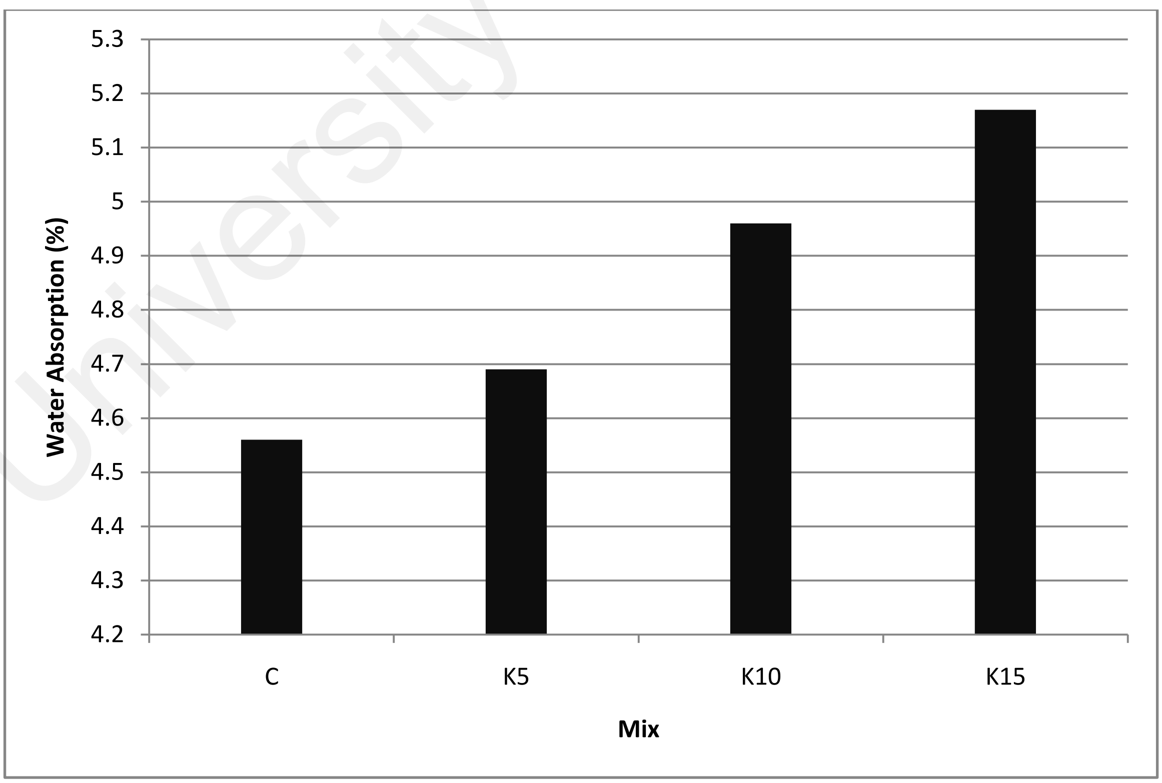


Figure 4.35: Water absorption of all kaolin mixes and control mix

4.6 Fresh properties of concrete

The fresh concrete test results of all mixes are presented and discussed in this section. The effects of type of pozzolan and filler powders, different coarse aggregate properties i.e. maximum size, grading and shape on flowability, passing ability and segregation resistance are presented. The slump flow, V-funnel, L-box, J-ring and segregation sieve tests were used to evaluate flowability, passing ability and segregation resistance.

The fresh densities of the mixes were determined and plotted in Figures 4.36, 4.37 and 4.38. The fresh densities of the mixes are within the range of 2324-2393 kg/m^3 and are used to assess the applied pressure on framework during casting. The difference in densities of mixes is a result of the different types of mineral admixture used and superplasticizer dosage of mixes. However, Figure 4.36 indicates that the fly ash mixes exhibit low values of density when compared with the control mixes due to the low specific gravity as well as low superplasticizer dosage required. From Figure 4.37, it is clearly observed that the mixes containing limestone powder produce high values of density when compared with the control mixes. This is attributed to the limestone having a high specific gravity compared to sand as well as the filling effect of limestone. However, all mixes produce dense mixes with insignificant differences between them.

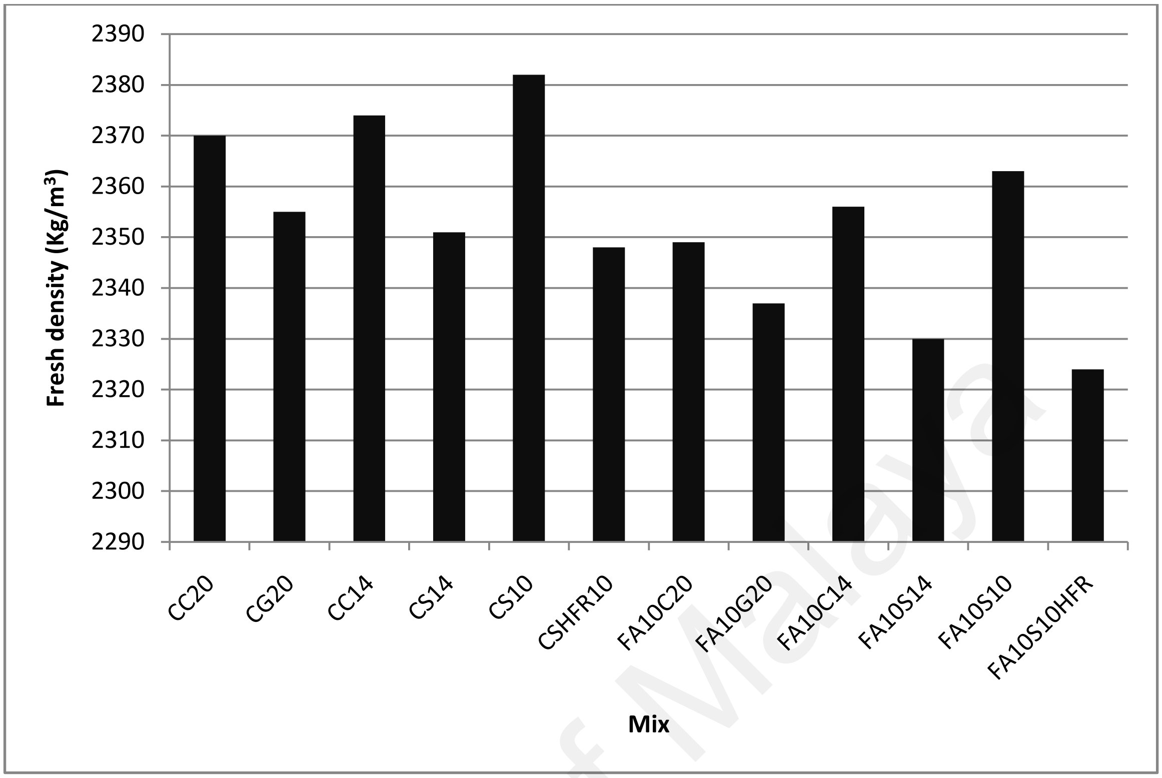


Figure 4.36: Fresh densities of fly ash and control concrete mixes

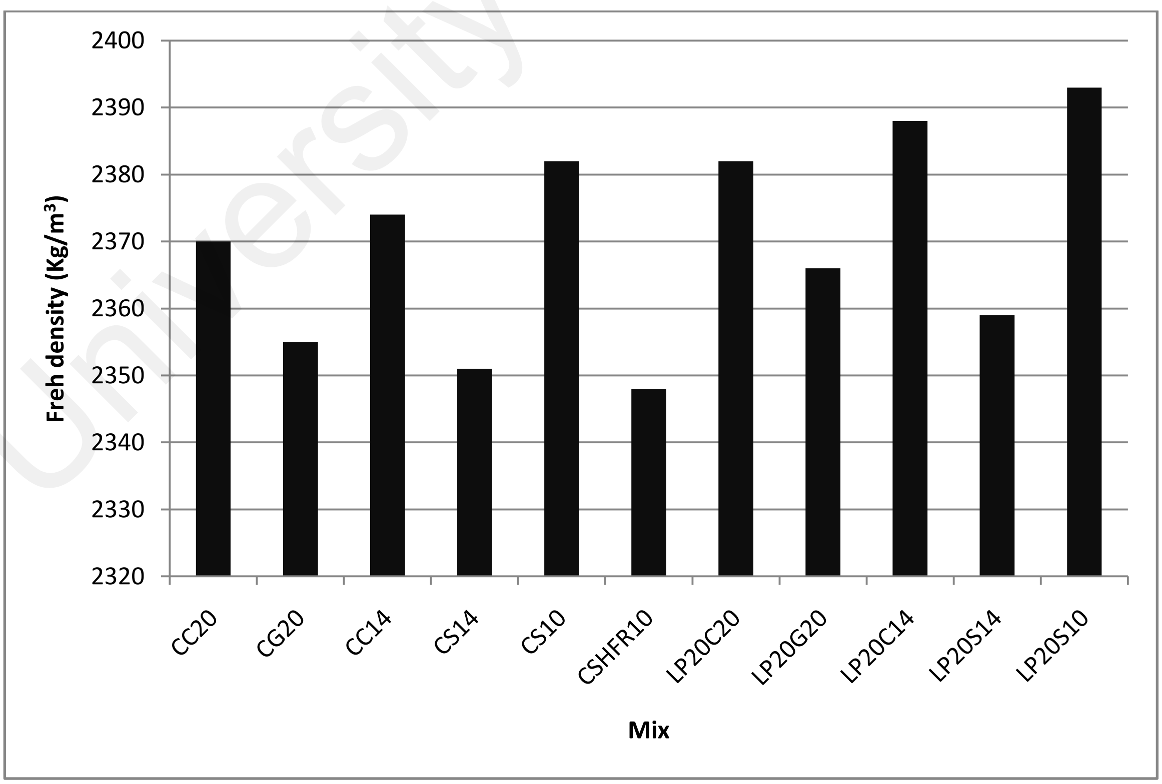


Figure 4.37: Fresh densities of limestone and control concrete mixes

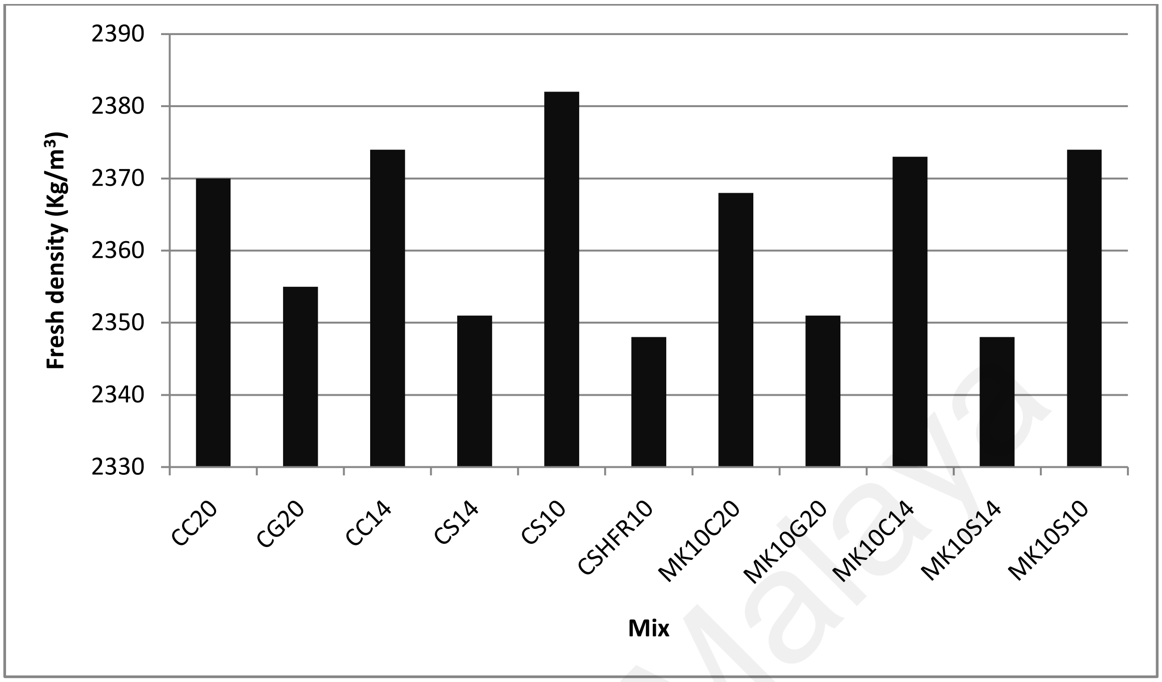


Figure 4.38: Fresh densities of metakaolin and control concrete mixes

4.6.1 Effect of mineral admixtures

The effect of powder mixes on the fresh properties are presented and discussed. Two types of flowability tests are conducted namely (slump flow and V-funnel). Two types of passing ability tests are conducted namely (L-box and J-ring tests). Moreover, one test is conducted to evaluate stability of fresh mixes namely (segregation sieve). The flowing ability of fresh concrete is described by slump flow investigated with Abrams flow. Table 4.4 shows the results of the slump flow diameter of all mixes at the same coarse aggregate content which varied between the ranges of 589 and 683 mm. The values of T_{50cm} represent the time required for the concrete flow to reach a spread of 50cm diameter, while the values of slump flow diameter represent the maximum spread. From the results all mixes satisfy the requirements of SCC as mentioned in section 2.6 with the exception of the mixes having aggregate with a high flakiness particle ratio. Thus, all mixes with the exception of those having aggregate with a high flakiness particle ratio are assumed to have good consistency and workability with regards to filling ability. However, these results display a wide range of variation. This variation illustrates the effect of coarse aggregate variables on the filling ability of SCC mixes as well as the effect of the type of mineral admixtures used.

Figure 4.39 shows the effect of pozzolan and filler on the flowability of concrete, red line represents the minimum level of slump flow diameter according to the requirements of SCC as mentioned in section 2.6 that is 600mm. The results show that the mixes containing fly ash exhibit higher slump flow diameter than the control and other mixes. This is attributed to the fact that the use of fly ash increases the paste content and reduces aggregate volume due to the low specific gravity of fly ash. As paste content increases the aggregate content decrease and thus the slump flow will increase (Saffiudin, 2008; Bonen and Shah, 2005; Yen et al., 1999). However, the metakaolin and limestone mixes have high slump flow diameter when compared with control mix due to higher paste content than the control mix.

The results of T_{50cm} of all mixes are presented in Figure 4.40. As observed from Figure 4.40 the lower T_{50cm} values of the fly ash mixes are due to the fact that fly ash mixes have good filling ability of their mortar mixes as well as a higher paste content and thus, contribute to lower T_{50cm} values. In contrast, the T_{50cm} of control mix is slightly lower than T_{50cm} of limestone mixes and T_{50cm} of metakaolin mixes.

The values of the flow time V-funnel test (T_f) represent the ability of the concrete to flow out of the funnel, while ($T_{f\text{-}5min}$) values represent the same ability but after refilling the funnel and allowing concrete to discharge after 5minutes following the refilling. Table 4.4 illustrates the results of the V-funnel test and the values are within the limits for SCC range between (6-12 seconds) and as mentioned in section 2.6. No blocking or segregation is observed for all mixes except for the mixes containing aggregate with a high flakiness ratio. The results clearly show the effect of mineral admixtures type on the ability of concrete to flow. Figure 4.41 reveals that fly ash mixes exhibit a low T_f and $T_{f\text{-}5min}$ when compared with other mixes for reasons identical to those mentioned above.

Table 4.4: Flowability results of all mixes

Symbol	Slump Flow test		V-funnel test	
	SFD (mm)	T_{50cm} (sec)	T_f (sec)	T_{f5min} (sec)
CC20	663	2.1	9	12
CG20	644	2.8	13	15
CC14	647	2.8	7	11
CS14	632	3.5	10	13
CS10	627	4	9	14
CSHFR10	589	7.3	19	22
FA10C20	683	1.2	8	11
FA10G20	666	2	11	13
FA10C14	668	1.8	6	10
FA10S14	657	2.7	10	12
FA10S10	648	3.2	6	9
FA10S10HFR	600	6.2	18	21
LP20C20	670	2.2	9	11
LP20G20	650	2.6	13	15
LP20C14	656	3	8	12
LP20S14	640	3.7	11	13
LP20S10	635	4.2	9	11.5
MK10C20	679	3	8	12
MK10G20	660	3.3	10	13
MK10C14	664	3	7	12
MK10S14	649	3.4	11	12
MK10S10	647	3.7	8	11

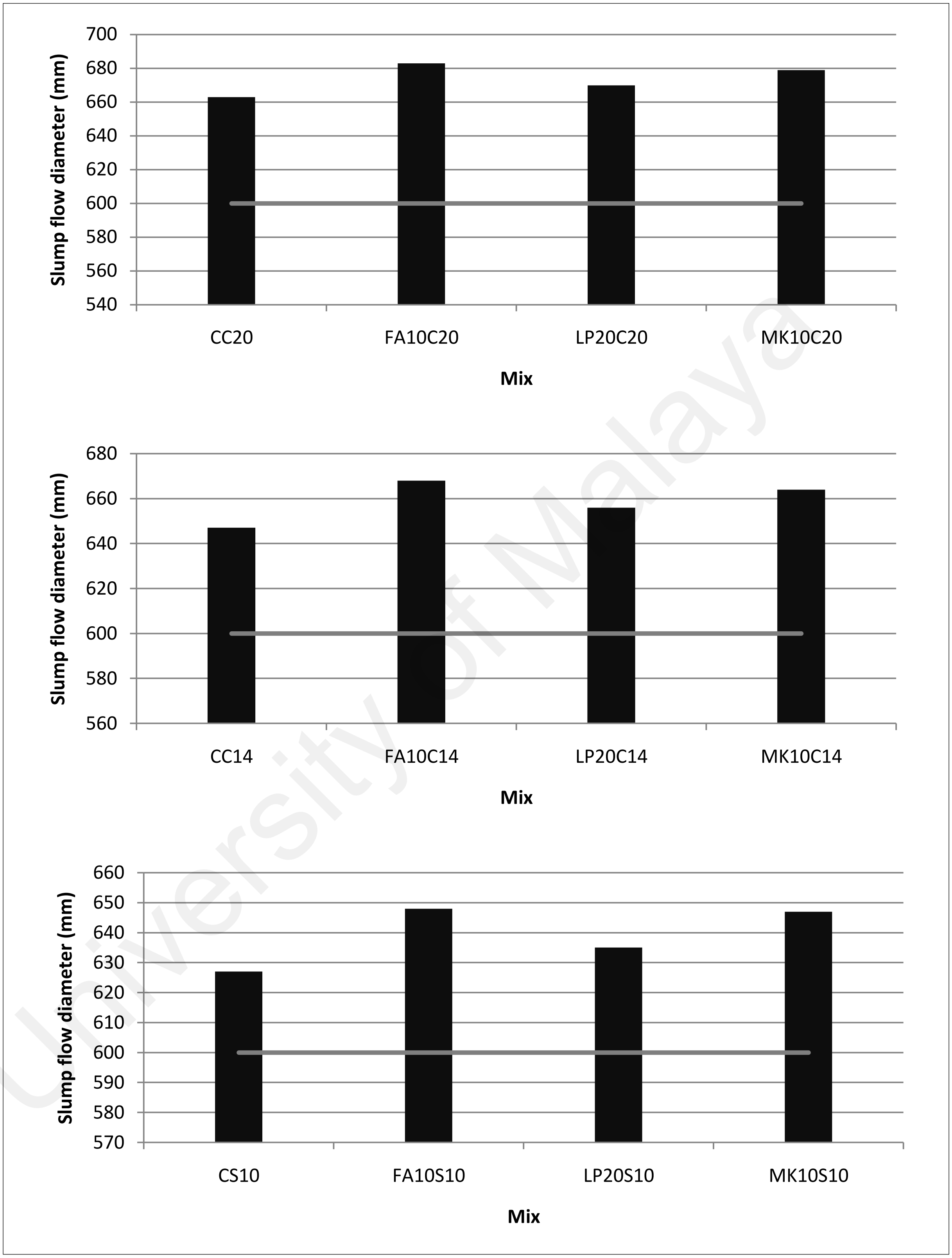


Figure 4.39: Slump flow diameter of different mineral admixture concrete mixes

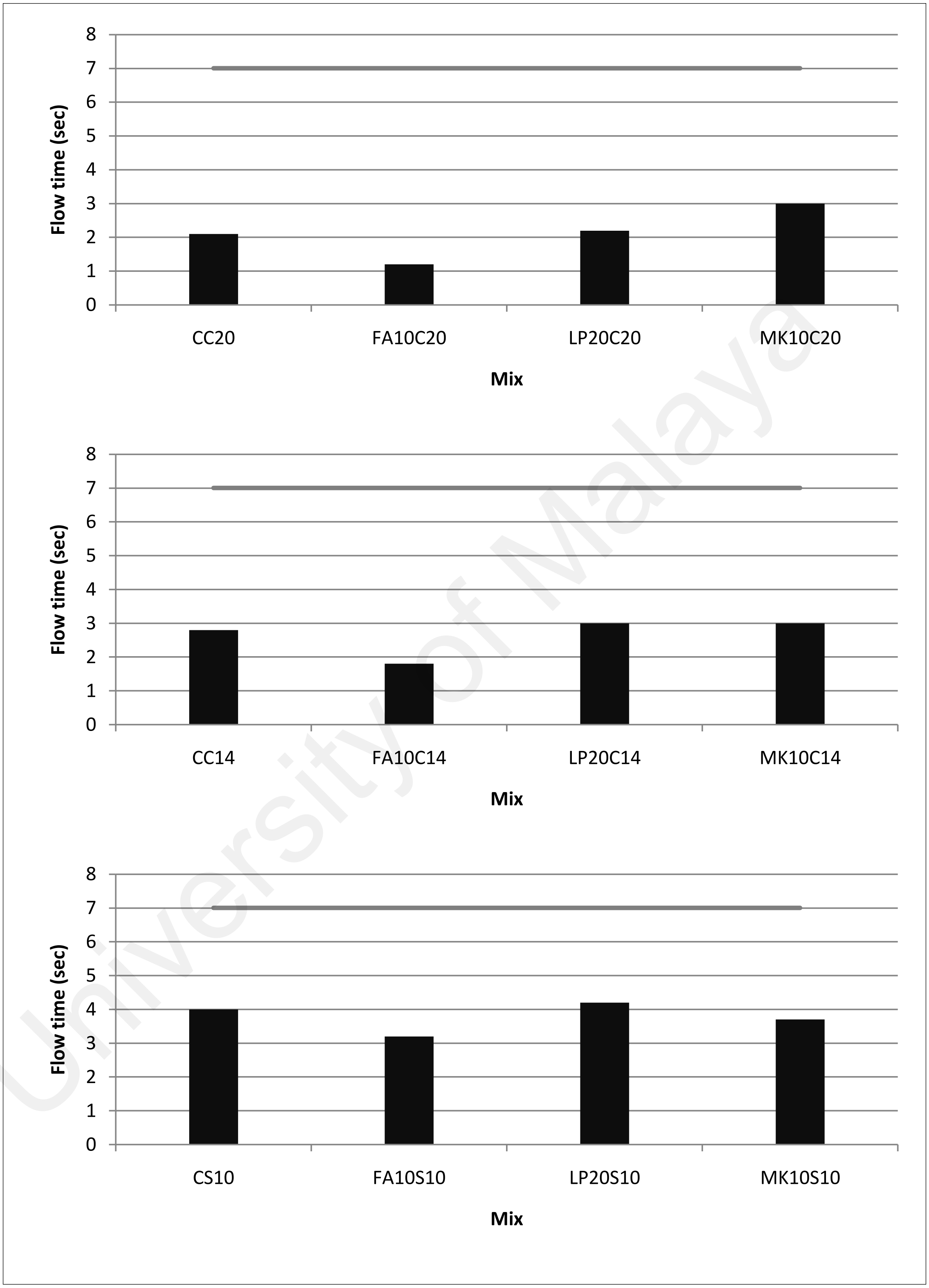


Figure 4.40: Flow spread time at 50cm of different mineral admixture concrete mixes

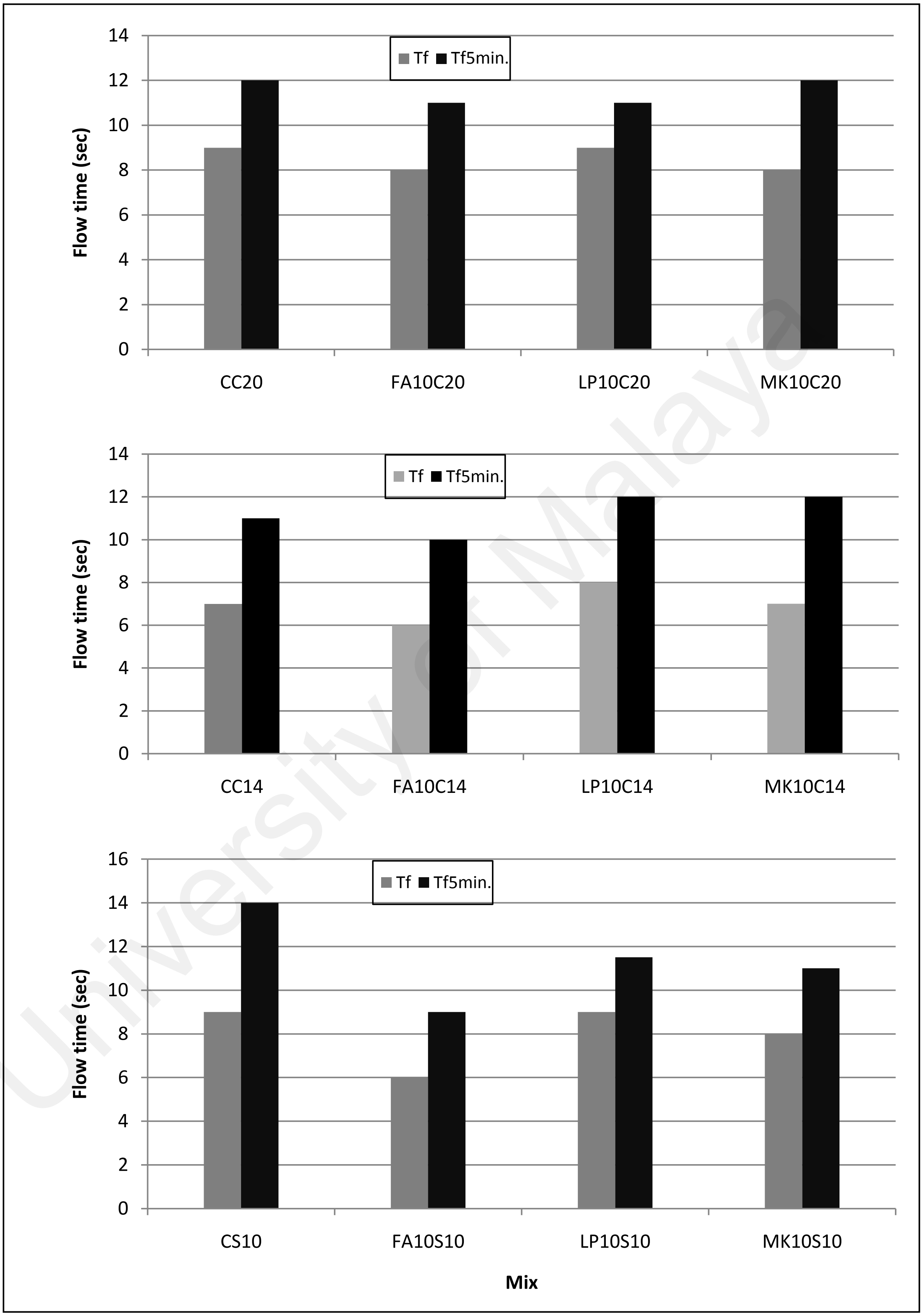


Figure 4.41: Flow time and flow time after 5min. of different mineral admixture concrete mixes

The results of passing ability of mixes are listed in Table 4.5 for L-box and J-ring tests and the values are within the limits for SCC as mentioned in 2.6 except the mixes with coarse aggregate containing high flakiness ratio. The SCC limits of blocking ratio of L-box test ranges between (0.75 to1.0) and height difference of J-ring test range between (0 to15mm). The effect of mineral admixtures on the passing ability is presented in Figures 4.42 to 4.45. The mixes containing fly ash exhibit good passing ability through steel bars in both J-ring and L-box tests. This could be due to the low aggregate ratio and high paste volume of fly ash mixes. Generally, as the aggregate volume decreases and paste volume increases, good passing ability is obtained as the concrete passes through the steel bar. The results shown in study of Turk and Kandemir (2010) indicated that the increase in the replacement level of fly ash led to the concurrent increase in blocking ratio of the mixes at the same mix proportions. The blocking ratio values of MK and LSP mixtures revealed slightly high values compared to that of control mixture. This behavior is due to high paste content in MK and LSP mixtures and reduced aggregate volume leading to high passing ability.

The values of T20 to T40 symbolize the concrete times to flow to distance 20, and 40 cm respectively. Figure 4.43 demonstrates that the incorporation of fly ash as a partial replacement by weight of cement leads to a decrease in T_{20cm} and T_{40cm} when compared with other mixes. This is due to the decrease in aggregate content and increase in the paste volume of these mixes which in turn leads to an increase in the passing ability through the steel bar as well as to lubricating effect of the fly ash.

As can be observed from Figure 4.44, the slump flow diameter of J-ring test ranges between 580 to 680mm for various mixes. The maximum reduction in the slump flow diameter in the presence of J-ring does not exceed 50mm in order to maintain good passing ability (Brameshuber & Uebachs, 2001 cited in Saffiudin, 2008). The slump flow diameter values of J-ring test reveal a good passing ability of the mixes except for mixes containing aggregate with a high flakiness ratio i.e. CSHFR10 and FA10S10HFR. The slump flow diameter of J-ring test to slump flow diameter of slump cone test ratios vary between 96 and 99.5% as shown in Table 4.5, which was also obtained by Saffiudin (2008).

The height difference of J-ring test results of different mineral admixture mixes are presented in Figure 4.45. The results revealed that values decrease with the use of pozzolan (FA and MK) and filler powder (LP) in the mixtures compared to only the cement mixtures. This is attributed to an increment in paste volume and a reduction in aggregate volume. This would reduce the confinement of aggregate behind the steel bar as well as the reduced friction between aggregate particles, and thus producing mixtures with high passing ability.

Table 4.5: Passing ability results of all mixes

Symbol	L-box			J-ring		
	Blocking ratio (BR) =H2/H1	T_{20cm} sec.	T_{40cm} sec	Height difference (ΔH) (mm)	Slump Flow (mm)	SFD of J-ring to SFD of slump cone (%)
CC20	0.82	3	6	12.2	640	96.5
CG20	0.75	3.4	7	14.8	618	96.0
CC14	0.86	2.4	5.6	10.6	633	97.8
CS14	0.74	3.6	7	13.9	624	98.7
CS10	0.86	2.1	4	9.4	623	99.4
CSHFR10	0.69	5.6	9.3	19.4	571	97.0
FA10C20	0.88	2	4.3	8.0	680	99.6
FA10G20	0.76	3	6.5	11.0	649	97.5
FA10C14	0.92	1.6	3.5	6.3	665	99.6
FA10S14	0.81	2.5	5.3	9.1	652	99.2
FA10S10	0.95	1.2	3	5.0	647	99.9
FA10S10HFR	0.73	5	7	17.2	579	96.5
LP20C20	0.83	2.8	4.5	10	669	99.8
LP20G20	0.77	3.5	6.6	13	643	99.0
LP20C14	0.89	2.3	4.2	7.7	652	99.4
LP20S14	0.77	3.3	4.7	10	632	98.8
LP20S10	0.91	1.9	4.2	6.2	632	99.5
MK10C20	0.84	2.4	4.7	10.5	667	98.2
MK10G20	0.76	3	6.2	12	652	98.8
MK10C14	0.88	2.1	4.2	7.1	660	99.4
MK10S14	0.78	3	5.7	11.1	635	97.8
MK10S10	0.90	1.9	4	6.5	644	99.5

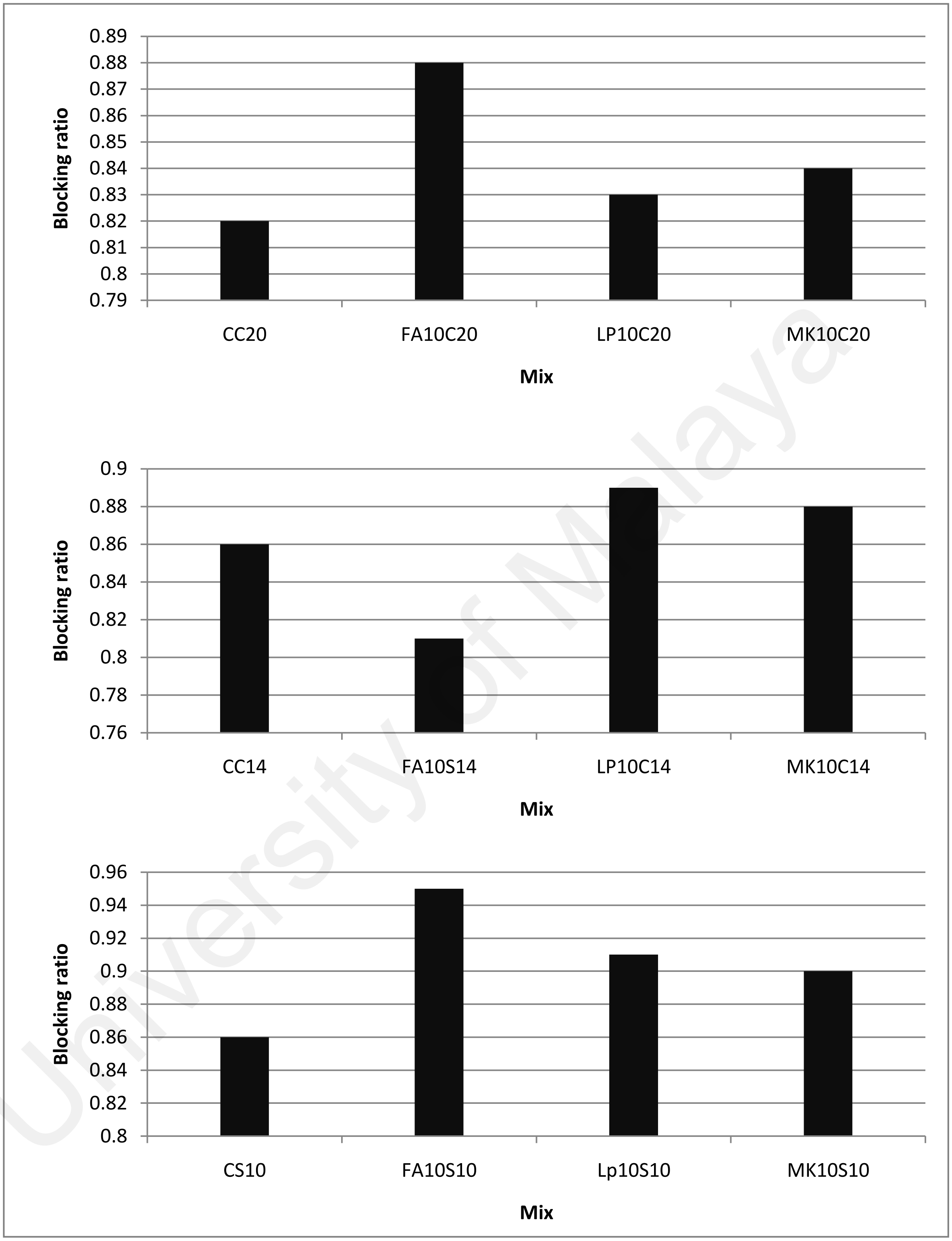


Figure 4.42: Blocking ratio of mixes with different mineral admixtures

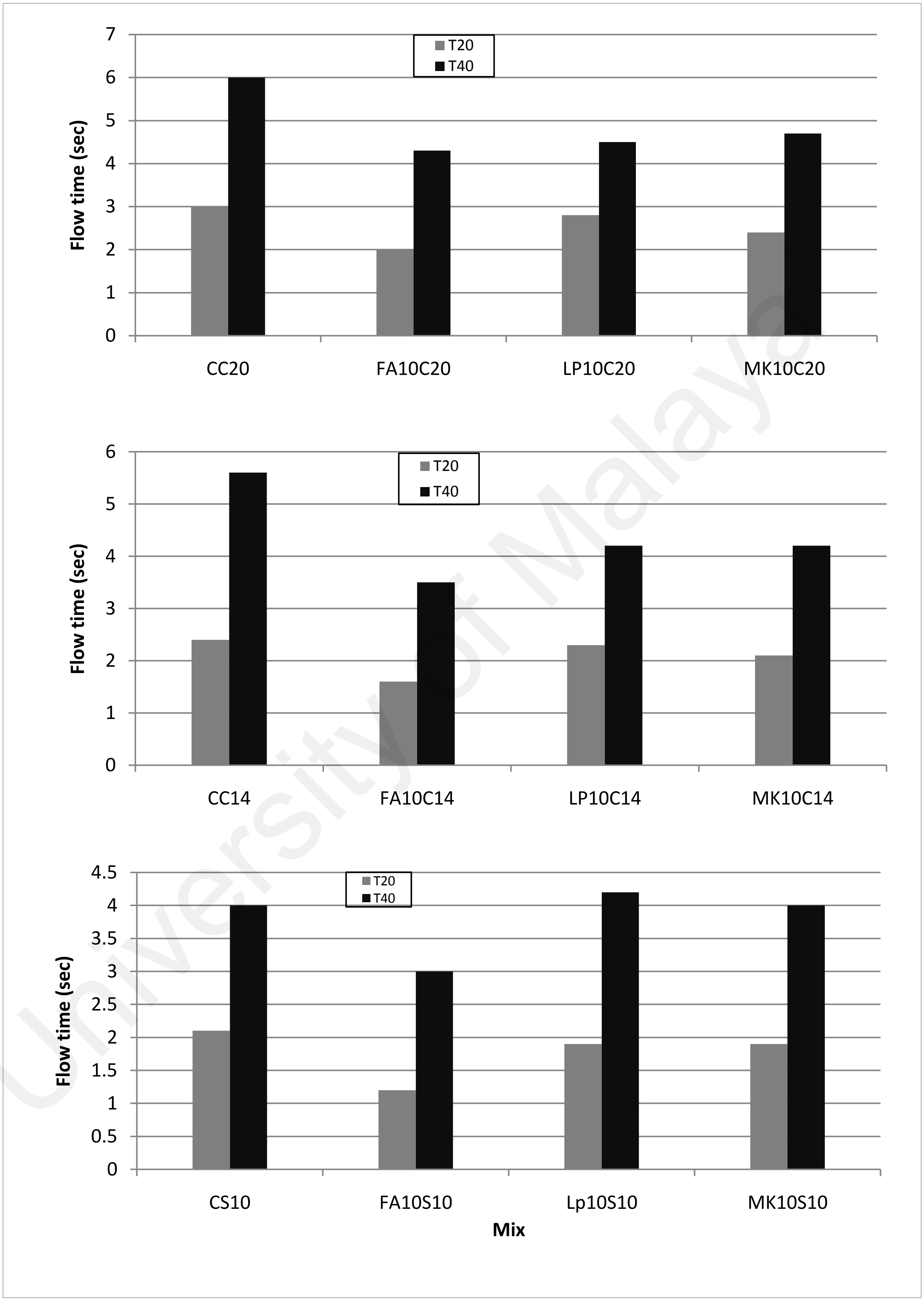


Figure 4.43: Flow times at 20 and 40 mm in L-box test of mixes with different mineral admixtures

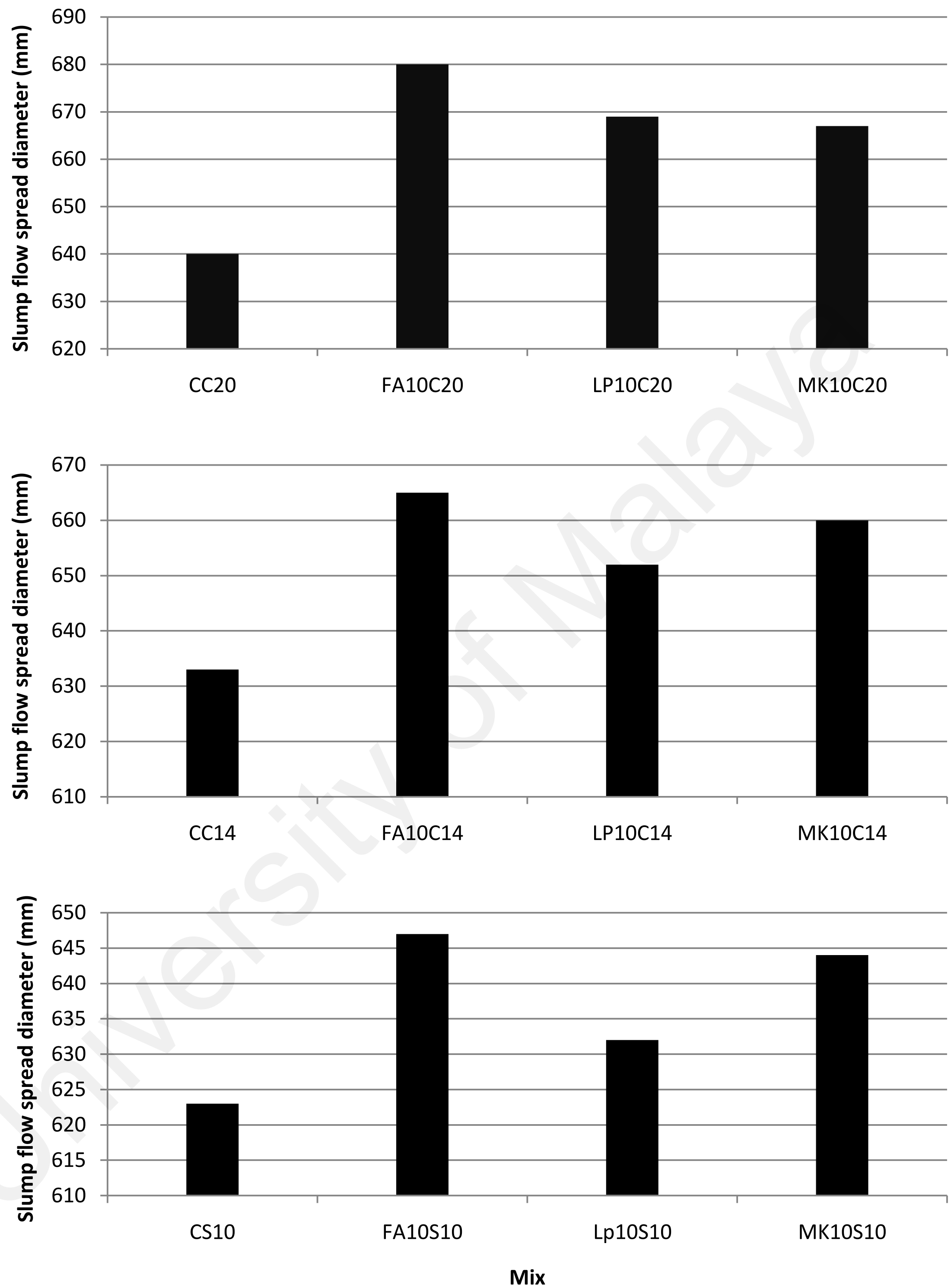


Figure 4.44: Slump flow diameter (J-ring test) of mixes with different mineral admixtures

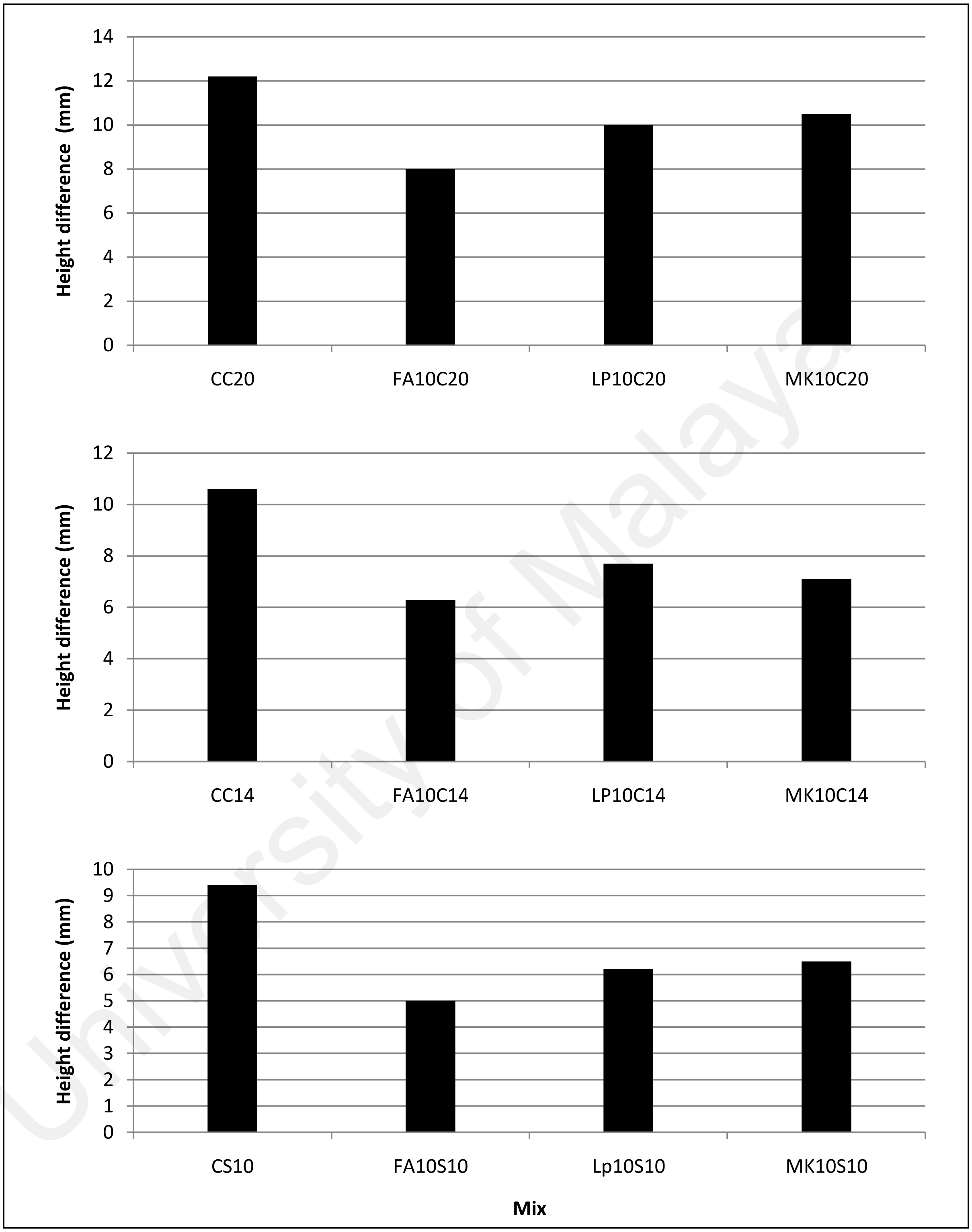


Figure 4.45: Height difference (ΔH) of mixes with different mineral admixtures

In this study, the segregation index test is adopted to estimate the static segregation resistance of concrete mixes. Table 4.6 shows the segregation index values of all mixes which vary between 5.5% and 19.4%. All the mixes are in the range of the specification limits which are between (5 to15%) except for mixes which contain aggregate with a high flakiness ratio, as mentioned in section 2.6. Generally, the mixes containing filler and pozzolanic materials exhibit a better static stability than control mixes as shown in Figure 4.46. This is attributed to the fact that the use of filler or pozzolanic materials leads to an increase in paste volume and a reduction of the aggregate volume. Indeed, the use of fillers and pozzolanic materials provide good plastic viscosity and the increase of the plastic viscosity reduces the settlement rate of coarse aggregate and thus increases the static stability of mixes.

Bouzoubaâ and Lachemi (2001) investigated the effect of high volume of class F fly ash on the fresh and hardened properties of self compacting concrete. This investigation showed that the presence of fly ash in the mixes led to increase in the stability of concrete (reduction in segregation index) in fresh state.

Libre et al. (2010) studied the effect of chemical and mineral admixtures on flowability and stability of self compacting concrete. Their study showed that the use of viscosity modifying admixture led to increase in viscosity of mixes and decrease in flowability and hence it would increase the superplasticizer dosage. While, the study observed that the use of fly ash led to increase flowability and enhance the viscosity of the mix without needing to using superplasticizer. In addition, the presence of limestone powder in the mixes led to improvement in the stability of the mixes without the need to increase superplasticizer dosage.

Table 4.6: Segregation index of all mixes

Mix No.	Segregation Index
CC20	12.9
CG20	14.8
CC14	10.6
CS14	13.9
CS10	9.4
CSHFR10	19.4
FA10C20	8.9
FA10G20	11.6
FA10C14	6.9
FA10S14	9.1
FA10S10	5.5
FA10S10HFR	17.2
LP10C20	10
LP10G20	12.9
LP10C14	7.3
LP10S14	10
LP10S10	6.2
MK10C20	10.5
MK10G20	12
MK10C14	7.1
MK10S14	10.1
MK10S10	6.5

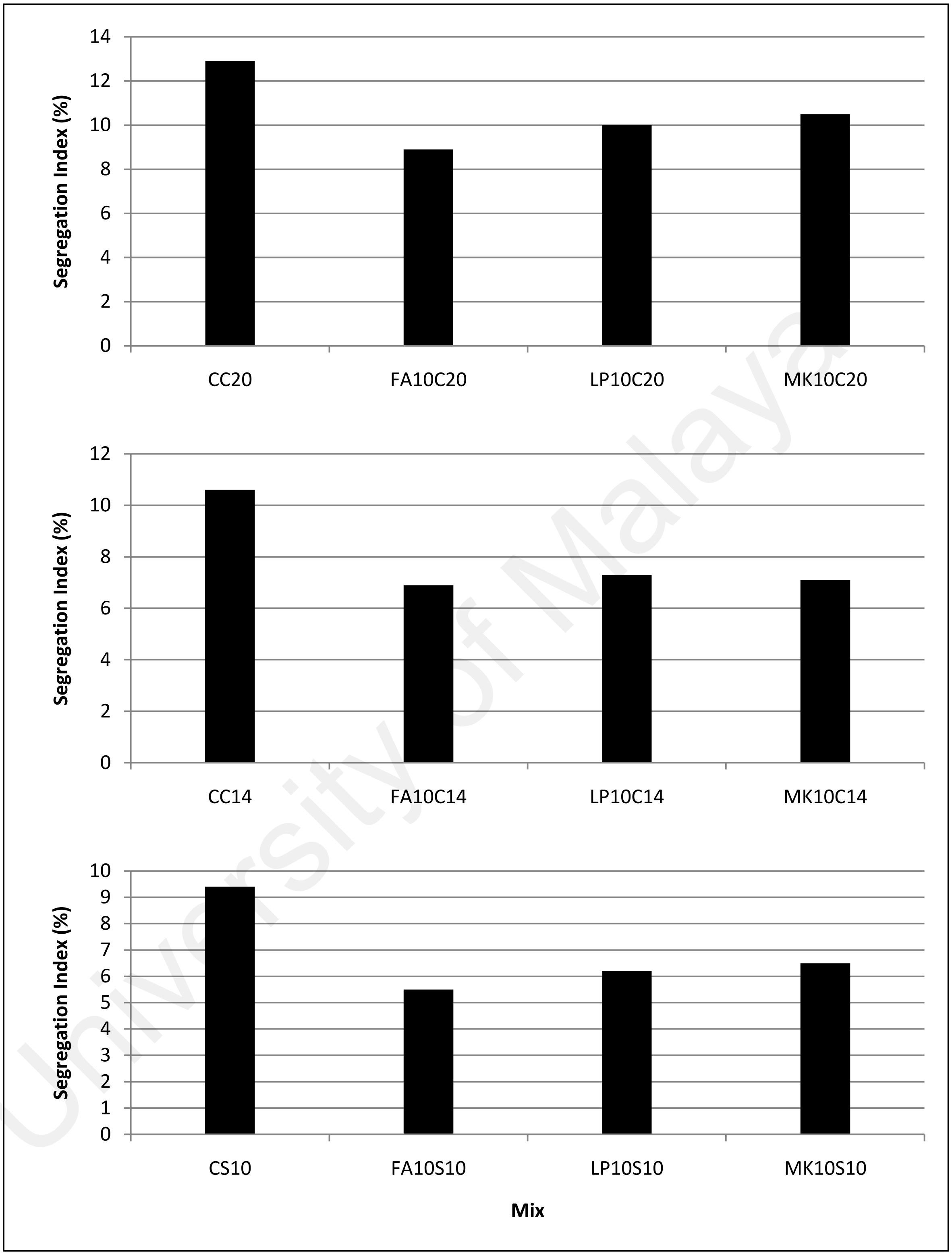


Figure 4.46: Segregation index (SI) of mixes with different mineral admixtures

4.6.2 Effect of coarse aggregate volume

The effect of different volume ratios of 20mm graded coarse aggregate on the fresh properties of control mix are presented and discussed. The results of slump flow diameter are varied according to volume ratio of coarse aggregate and are shown in Figure 4.47. At the same mix proportion of mortar, an increase in slump flow diameter was observed when the volume ratio of coarse aggregate was reduced. The increment in coarse aggregate volume leads to increasing the numbers of particles, which in turn increases the probability of the collision of these particles thus increasing friction between particles. A high friction of aggregate particles causes an increase in the viscosity and yield stress of fresh concrete. A high amount of mortar in the concrete causes it to behave as a lubricated material between aggregate particles and whilst also modifying the flow ability of fresh concrete. Similar trends have also been observed by Khayat (1999a) and ACI report (2008). Khayat (1999a) also reported that to get good deformability it is necessary to reduce coarse aggregate content.

However, Figure 4.48 shows that T_{50cm} increases with increase in coarse aggregate content due to the same reasons mentioned before. Indeed in these aforementioned cases high content of coarse aggregate would reduce the excess mortar around aggregate particles and thus would increase friction which results from the collision of aggregate particles due to the reduced gap between them. Hu and Wang (2011) concluded that at the same mix proportion of mortar, the increase of coarse aggregate content leads to increase in viscosity and yield stress of concrete.

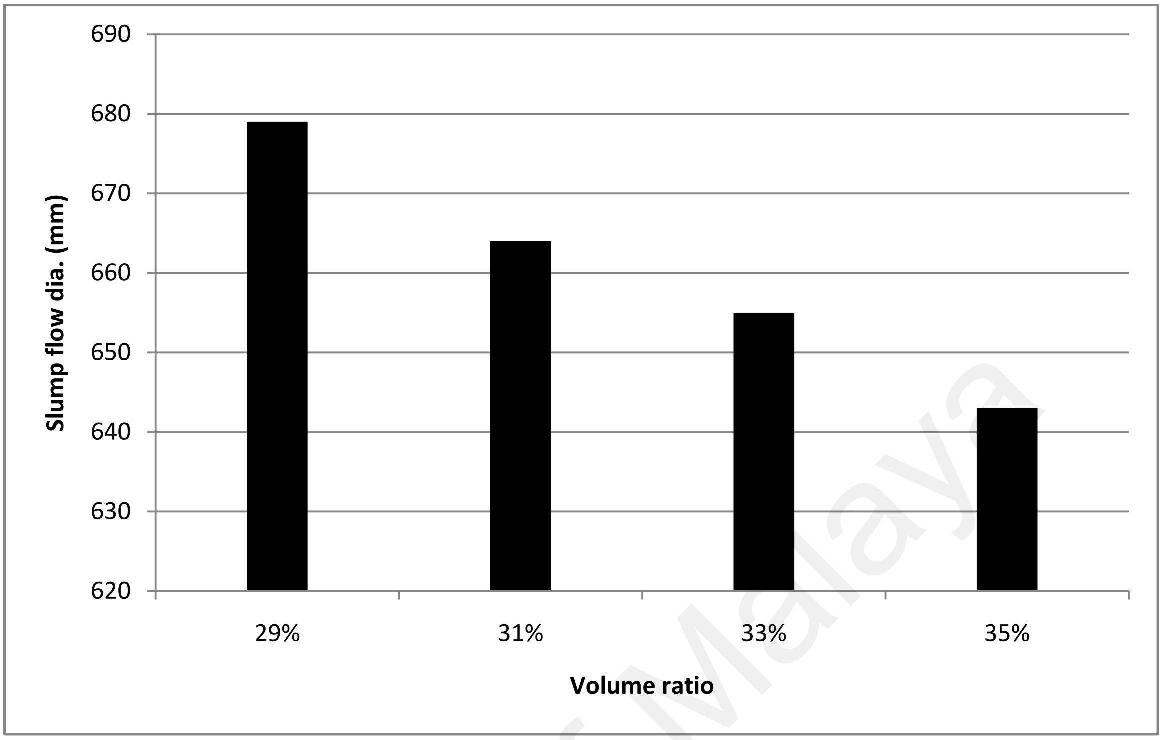


Figure 4.47: Slump flow diameter of different coarse aggregate volume control mixes

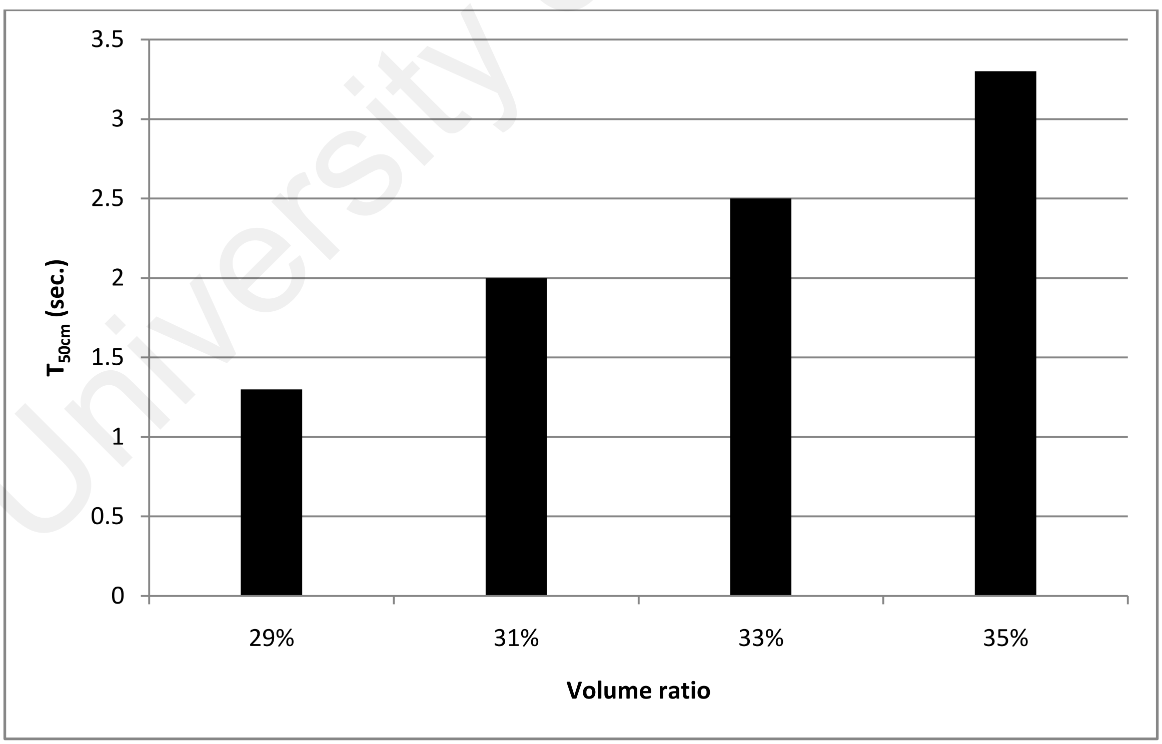


Figure 4.48: Flow time at 50cm of different coarse aggregate volume control mixes

The effect of coarse aggregate content on the T_f and T_{f5min} is shown in Figure 4.49. The increase in coarse aggregate in concrete leads to increment in T_f and T_{f5min}. This is in accordance with results of a study carried out by Rahim (2005). This is due to the increase in aggregate particles in the concrete which need to pass through the V-funnel orifice. Furthermore, the friction between particles increases as a result of collision of these particles.

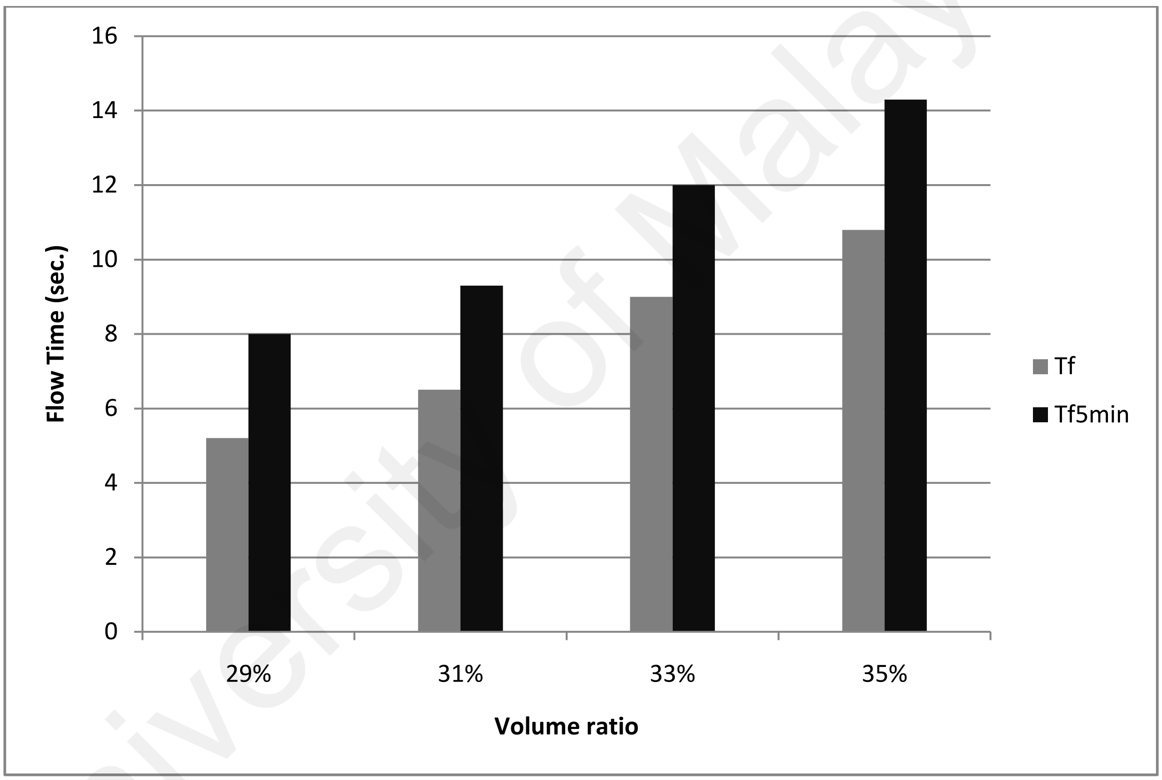


Figure 4.49: Flow time and flow time after 5min. of different coarse aggregate volume control mixes

The results of blocking ratio vary according to coarse aggregate content in the concrete mixes. As shown in Figure 4.50. It is clearly evident that the increment in aggregate content leads to a decrease in blocking ratio values due to confinement of some aggregate particles behind reinforcement bar. In addition, the effect of aggregate content on T_{20cm} and

T_{40cm} is shown in Figure 4.51. The results indicate that as aggregate content increases there is an increase of both of T_{20cm} and T_{40cm}.

The effect of aggregate content on the height difference (ΔH) inside and outside J-ring of fresh concrete is presented in Figure 4.52. Figure 4.52 indicates that the height difference results (ΔH) of mixes vary with different coarse aggregate contents. High coarse aggregate content leads to increase in ΔH. This increment in ΔH may be attributed to the fact that as concrete flows out from the cone, it would be blocked by the steel bar of the J-ring, resulting in the entrapment of some concrete.

The results of segregation index of fresh concrete vary according to the coarse aggregate content in the mixture, as shown in Figure 4.53. It reveals that a high amount of coarse aggregate in the mixes is more likely to result in segregation than. Therefore, it is clear that the mix which contains a high percentage of coarse aggregate has higher segregation index when compared to mixes which contain low percentages of coarse aggregate. The increase in coarse aggregate volume in the mixes means low mortar volume which in turn reduces the surrounding mortar layer of aggregate particles. Okamura (1997) concluded that one of the important factors to achieve self compactability is to reduce coarse aggregate volume in the mixes and in turn would reduce the probability of collision of particles and thus would improve flowability, passing ability and stability of fresh concrete mixes. Newman and choo (2003) mentioned in their study that high volume of coarse aggregate in the mixes lead to blockage of concrete flow and reduces segregation resistance of the mix.

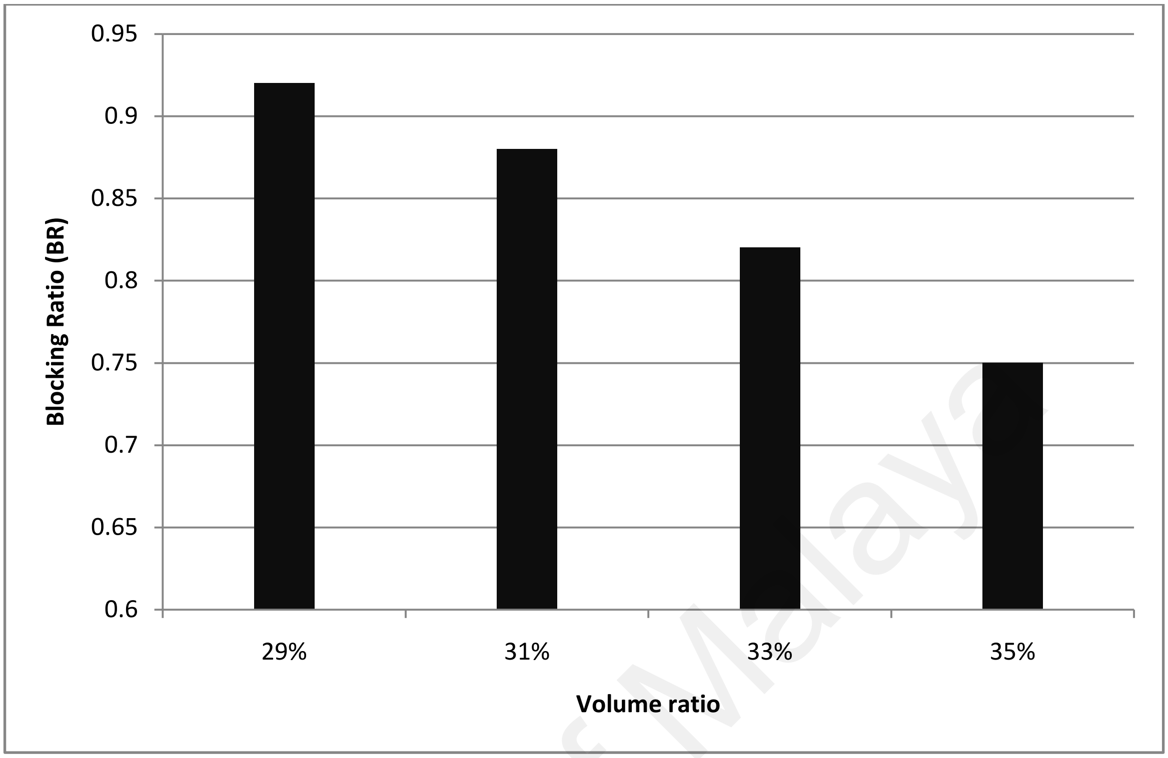


Figure 4.50: Blocking ratio of different coarse aggregate content

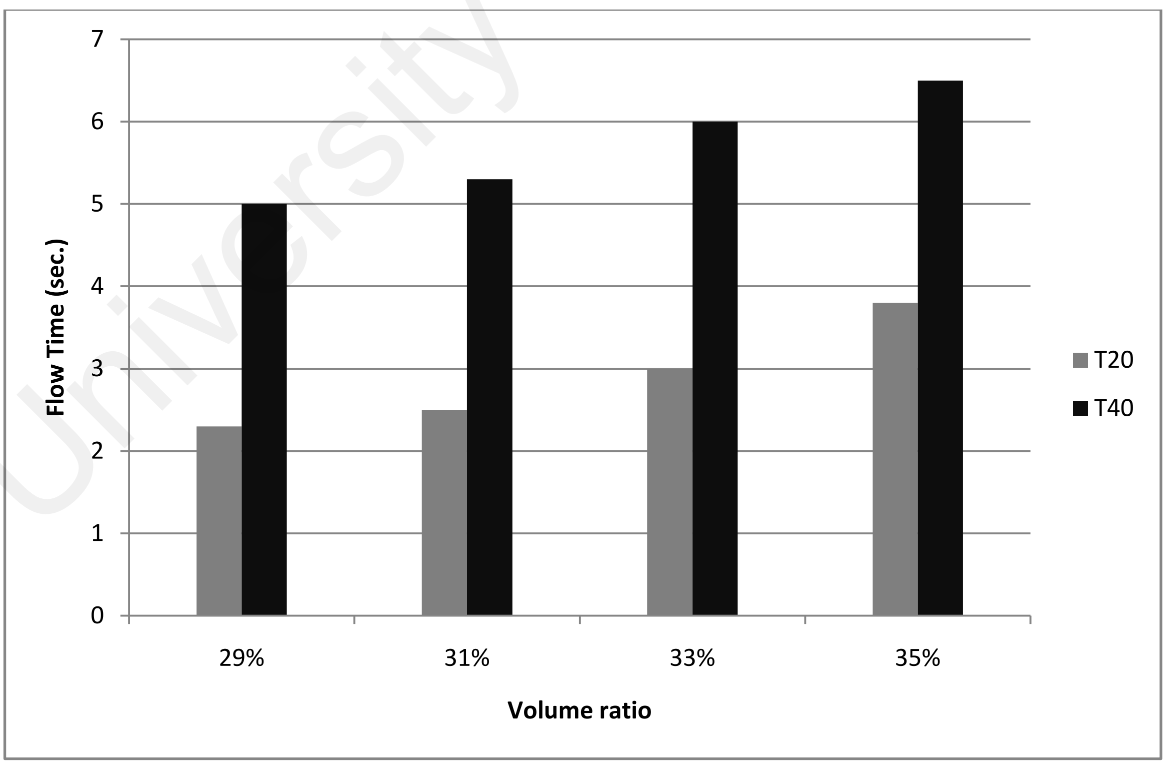


Figure 4.51: Flow time at 20cm & 40cm of different volume ratio of coarse aggregate

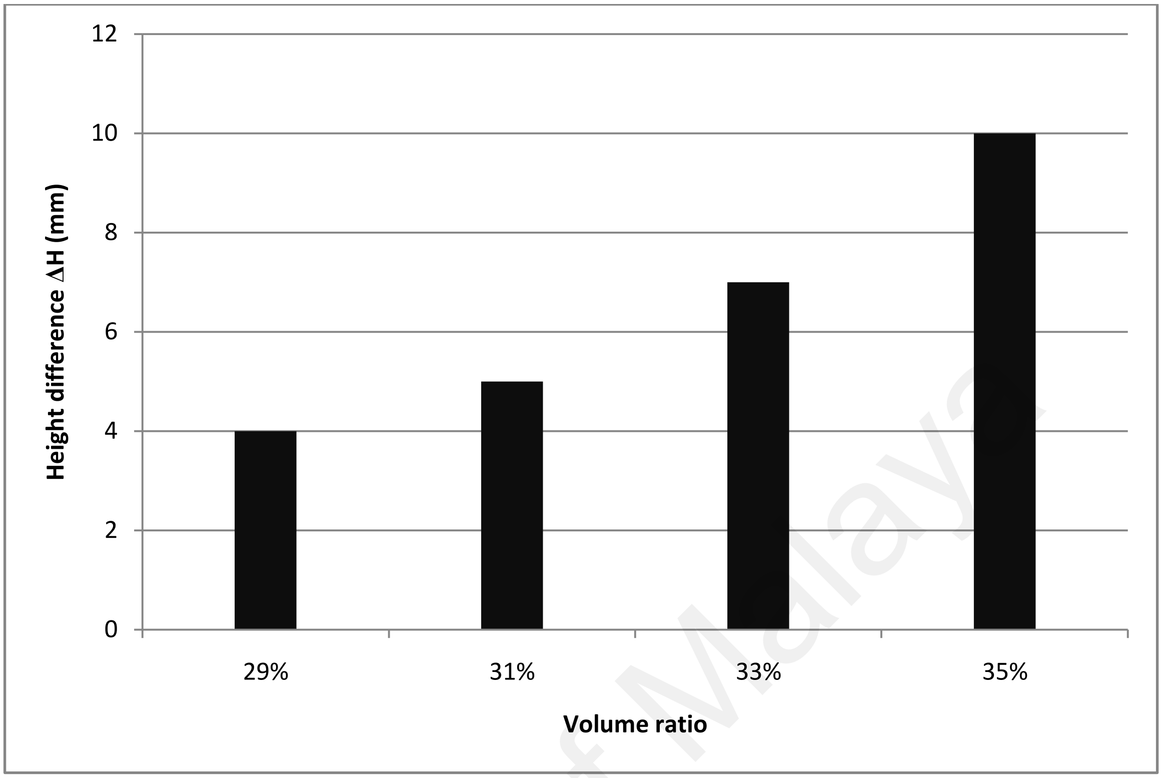


Figure 4.52: Height difference (ΔH) of J-ring test of different coarse aggregate content mixes

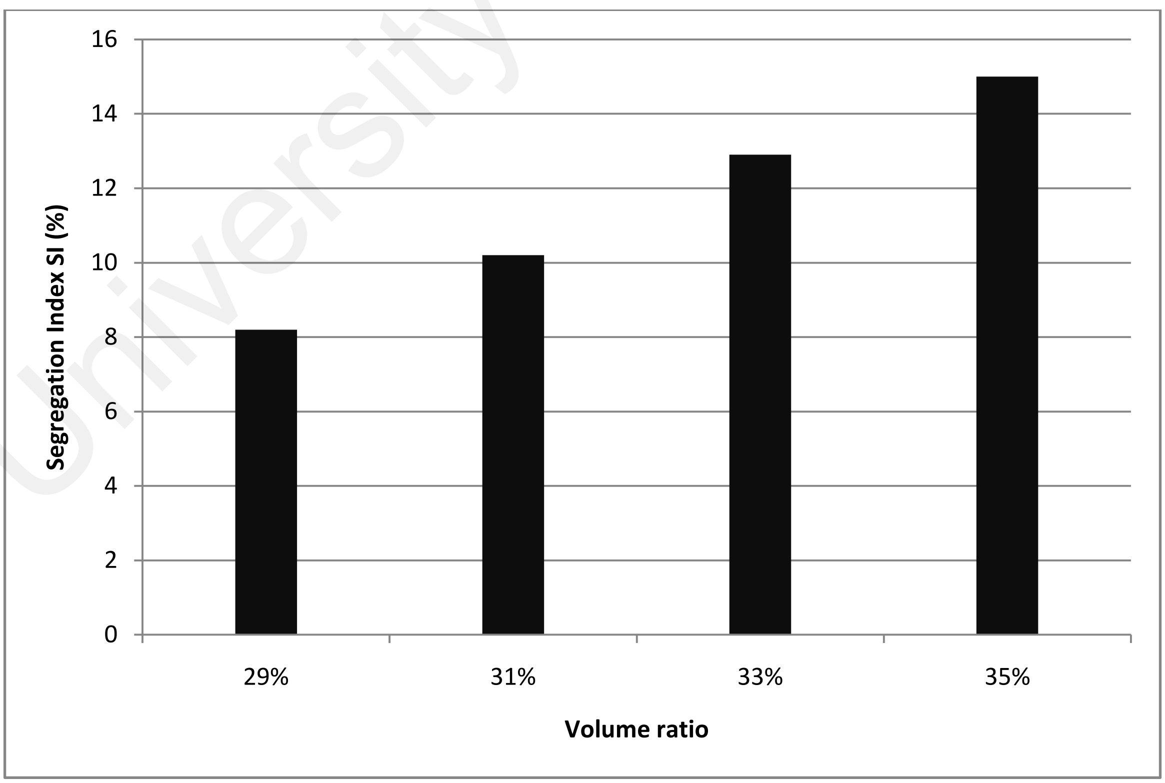


Figure 4.53: Segregation index of different coarse aggregate content mixes

4.6.3 Effect of coarse aggregate maximum size

The flow ability is influenced by maximum size aggregate as shown in Figure 4.54. The slump flow diameter (SFD) ranges between 627 and 683mm. Figure 4.54 reveals the effect of maximum sizes of 20mm and 14mm on the flow ability of mixes at the same grading. All the results in Figures 4.54 demonstrate that the slump flow diameter of fresh concrete increases when using larger maximum size coarse aggregate in the mixes. This agrees with the findings of Washa (1998a), Hu (2010) and Youjun et al. (2002). This may be due to using smaller particles leads to an increase probability of collision among particles. This increases the friction between particles and thus decreases flow ability of concrete. Hu and Wang (2011) reported in their study that the yield stress and viscosity of fresh concrete reduces with an increase in the maximum size of aggregate. This could be interpreted as a need for the lower maximum size to have a higher proportion of paste or mortar to cover it than larger maximum size. Indeed, mixes of concrete with the same mix proportion of both maximum sizes result in higher viscosity and yield stress.

Youjun et al. (2002) concluded that the flow ability and the velocity of flow increased with increased maximum size of coarse aggregate. This may be due to the fact that increasing the specific area of coarse aggregate leads to the need for more of the required paste in order to wrap the aggregate particles, after which the excess paste will decrease.

The results of flow time T_{50cm} are shown in Figure 4.55; these results indicate that as maximum size of aggregate increases, T_{50cm} is reduced. This is owing to the same reason as shown for slump flow results. The effect of aggregate on flow ability is occasionally not obvious due to combining more than one type of aggregate.

Figure 4.56 indicates that (T_f & T_{f5min}) values increase with an increase in the maximum size of coarse aggregate. This is in agreement with studies carried out by Okamura and Ouchi (2003), Rahim (2005), Khaleel (2011), and Krishna et al. (2010). They do however contradict the results of T_{50mm}. In this test, the fresh concrete which contains large aggregate particles looses more energy during its movement through the constricted end of the V-funnel box when compared to using smaller aggregate particles in concrete. This does not occur in the slump flow test with the wide open end of the slump cone.

Rahim (2005) used coarse aggregate with two maximum size of 10 and 20mm. This study found that SCC mixes containing 10mm coarse aggregate showed good flowbility compared to that of 20mm.

Krishna et al. (2010) found that the presence of small maximum size of coarse aggregate in the mixes significantly enhanced the flowability of V-funnel compared to large maximum size.

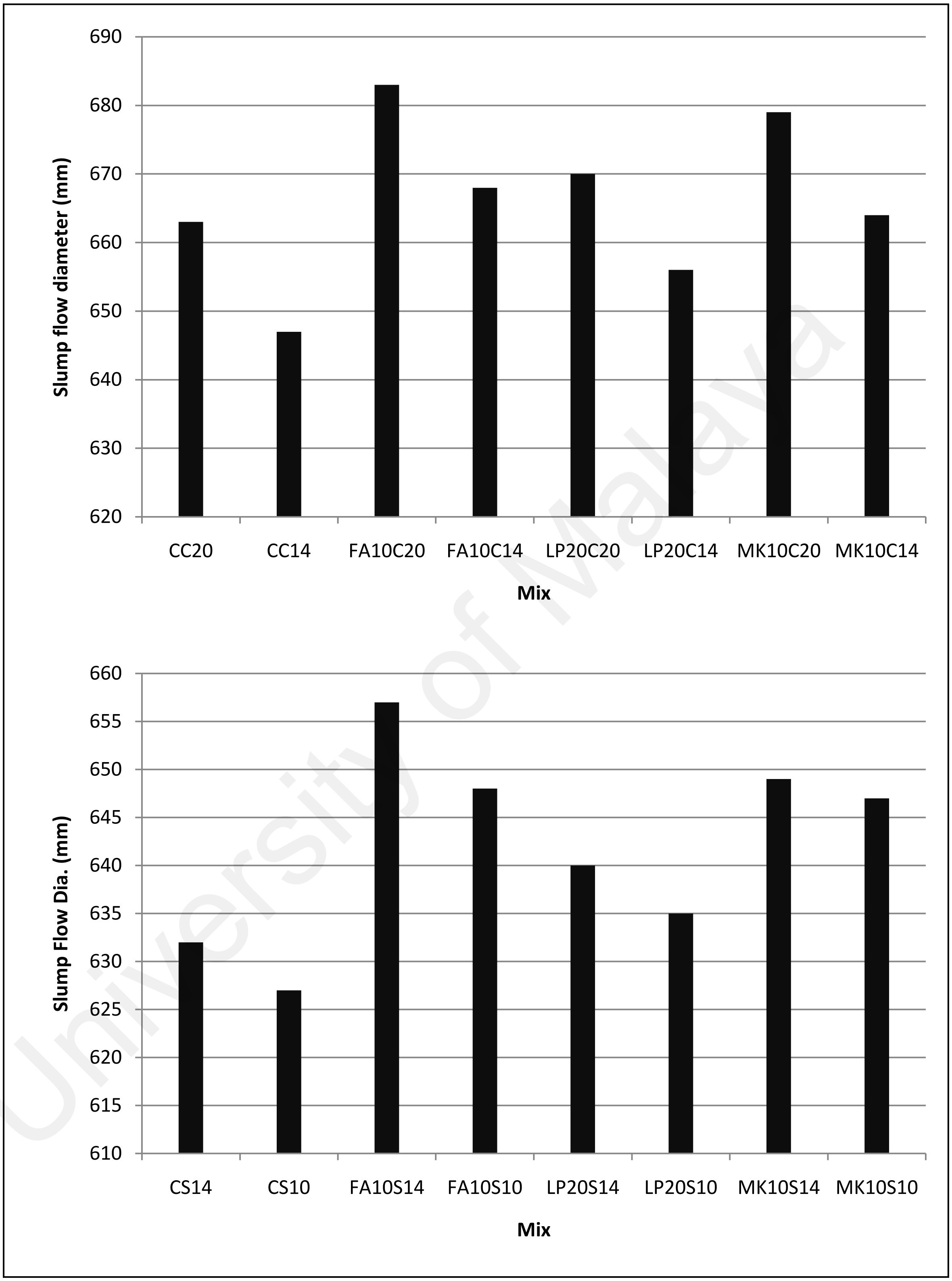


Figure 4.54: Slump flow diameter of different maximum size aggregate mixes

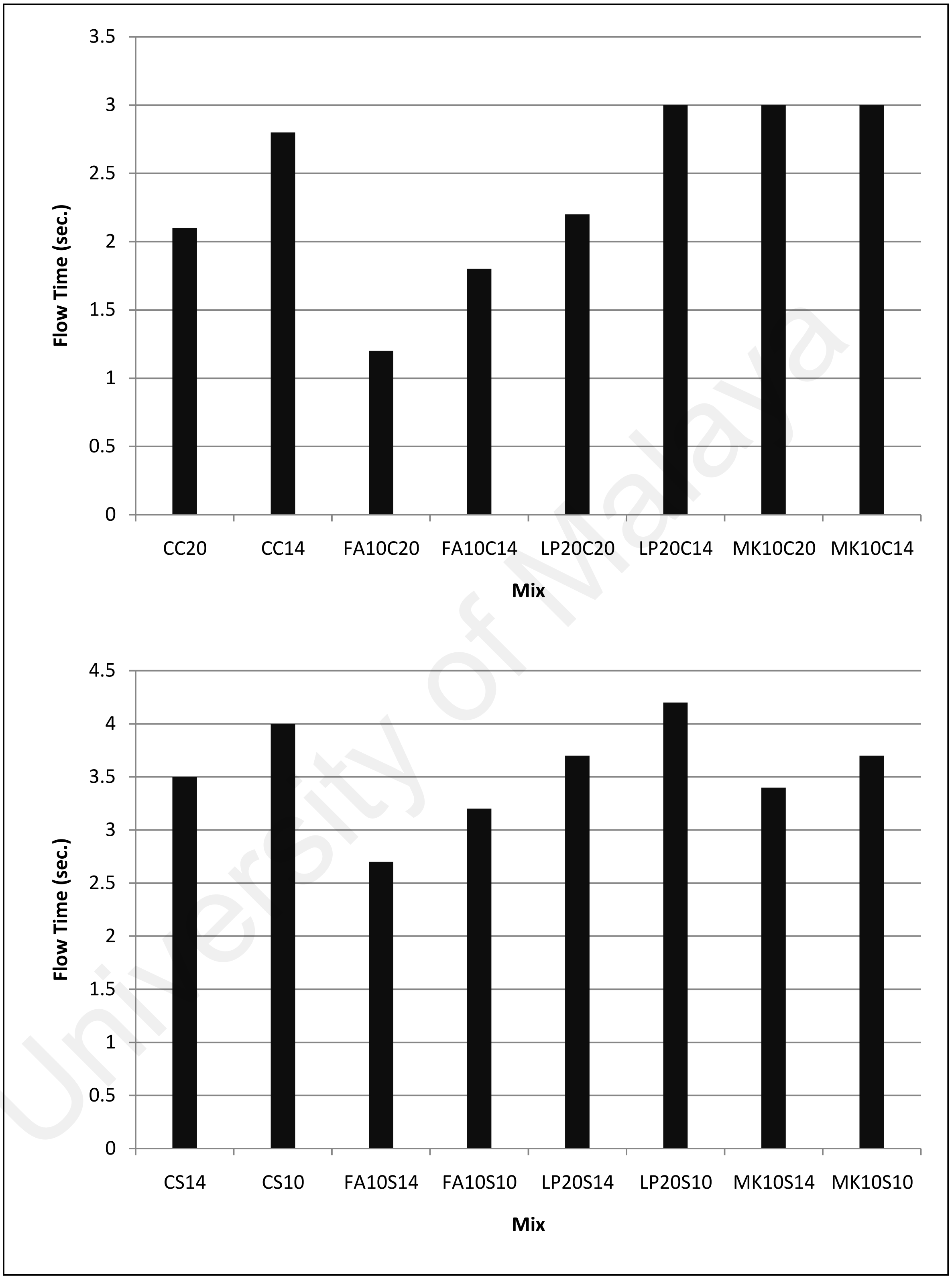


Figure 4.55: T_{50cm} of different maximum size aggregate (14 &10mm single) mixes

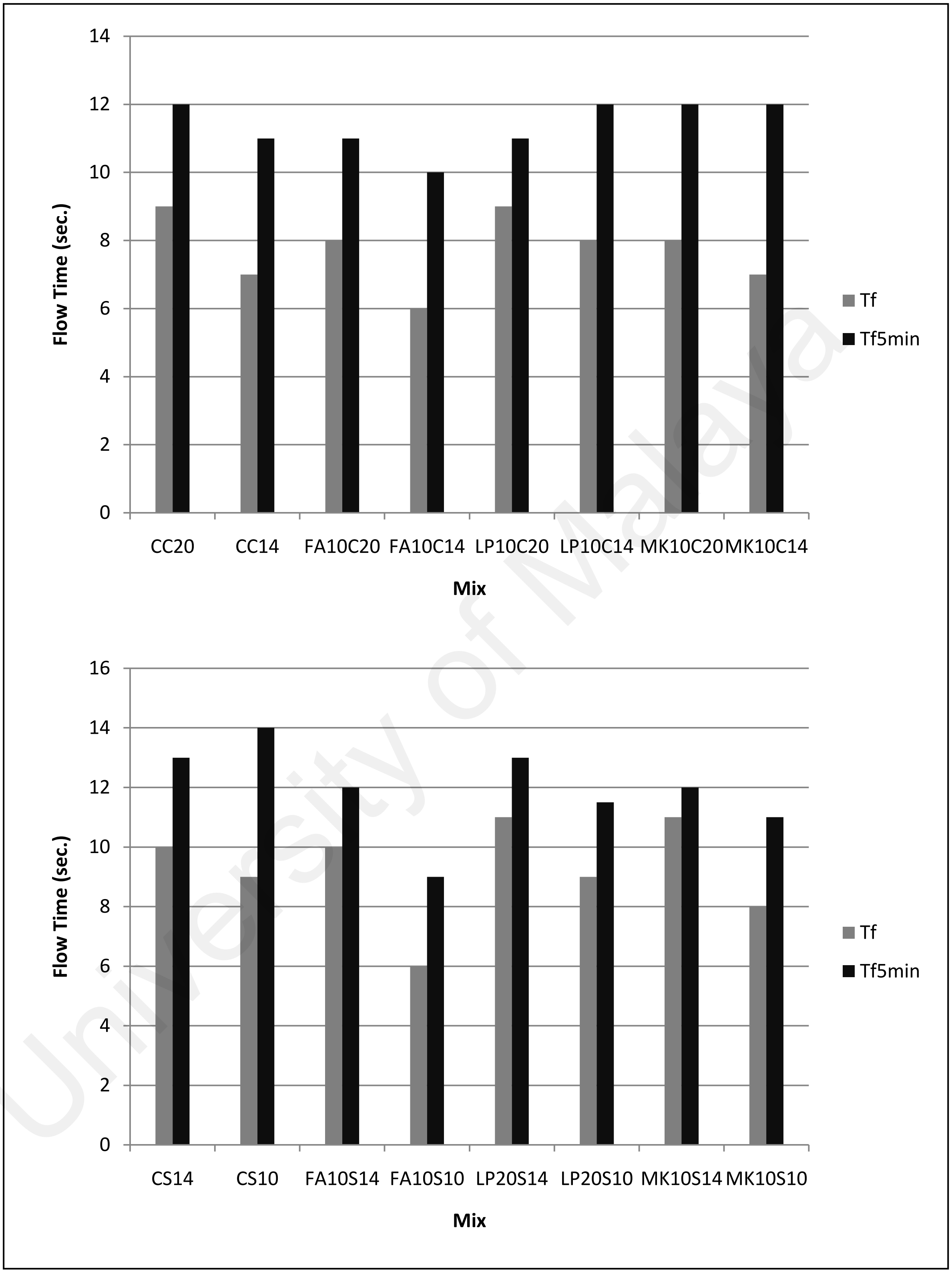


Figure 4.56: T_f & T_{5min} of different maximum size aggregate mixes

The effect of maximum size aggregate on blocking ratio is demonstrated in Figures 4.57 and 4.58. Figures 4.57 and 4.58 show that the mixes containing 20mm aggregate lead to a decrease in blocking ratio value and an increased T_{20mm} and T_{40mm} when compared with the mixes contained 14mm aggregate. In addition, the 14mm single sized aggregate mixes have lower blocking ratio value compared to the 10mm single sized aggregate mixes as shown in Figure 4.57. Moreover, Figure 4.58 shows that the mixes with 10mm and single sized aggregate give lower T20mm and T40mm values than the mixes with 14mm and single sized aggregate. This may be due to the fact that the large maximum size tends to restrict the flow of fresh aggregate between steel bars. Conversely, the use of small maximum size aggregate in the mixes means that it passes freely through the steel bars of the L-Box. This was also observed by Rahim (2005).

Figure 4.59 illustrates that mixes which contain 20mm graded aggregate increase the height difference ΔH when compared with 14mm graded aggregate. Moreover, Figure 4.59 indicates that 14mm single sized aggregate mixes give higher ΔH values than 10mm single sized aggregate mixes. This is due to larger particles being prone to retention behind the ring of bars and thus reducing the concrete passing between them. Figure 4.60 shows the results of slump flow diameter of mixes in J-ring test. Generally, the mixes containing larger maximum size aggregate show less slump flow diameter.

Petersson (2001) showed that 10mm maximum size aggregate in the mixes leads to better passing ability than 20mm maximum size aggregate. This study also indicated that it is possible to use 10mm and 20mm maximum size aggregate to produce SCC.

The effect of maximum size of coarse aggregate on the results of segregation index is plotted in Figure 4.61. The mixes containing 20mm maximum size and graded aggregate exhibit less stability than the mixes containing 14mm maximum size and graded aggregate.

Indeed, in this case the segregation index values of 20mm graded aggregate mixes are higher than that of 14mm graded aggregate mixes. Furthermore, the mixes which contain 14mm single sized aggregate gave higher segregation index values than the mixes which contain 10mm single sized aggregate. From this it could be hypothesised that the smaller particles of coarse aggregate lead to a decrease in the difference of velocity between flowing the particles and an increase in homogeneity of distribution of aggregate particles. This subsequently increases the stability of the mixes as verified by Xie et al. (2005). Hu and Wang (2011) showed in the study that the use of smaller maximum size in mixes results in higher viscosity and yield stress.

Saak et al. (2001) reported that lower maximum coarse aggregate size lead to reduction in the segregation risk of aggregates due to reducing the difference in density between paste and the aggregate.

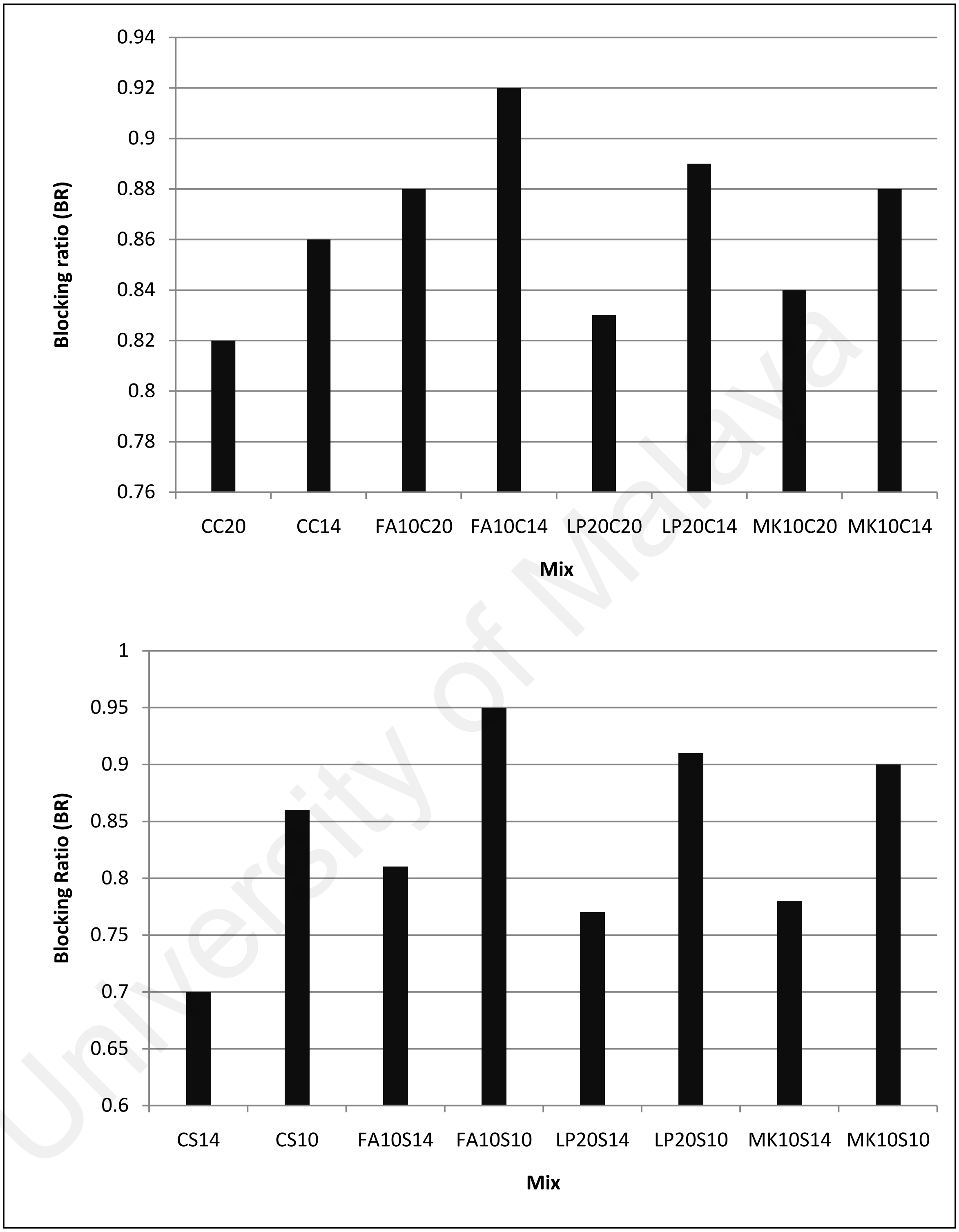


Figure 4.57: Blocking ratio different maximum size aggregate mixes

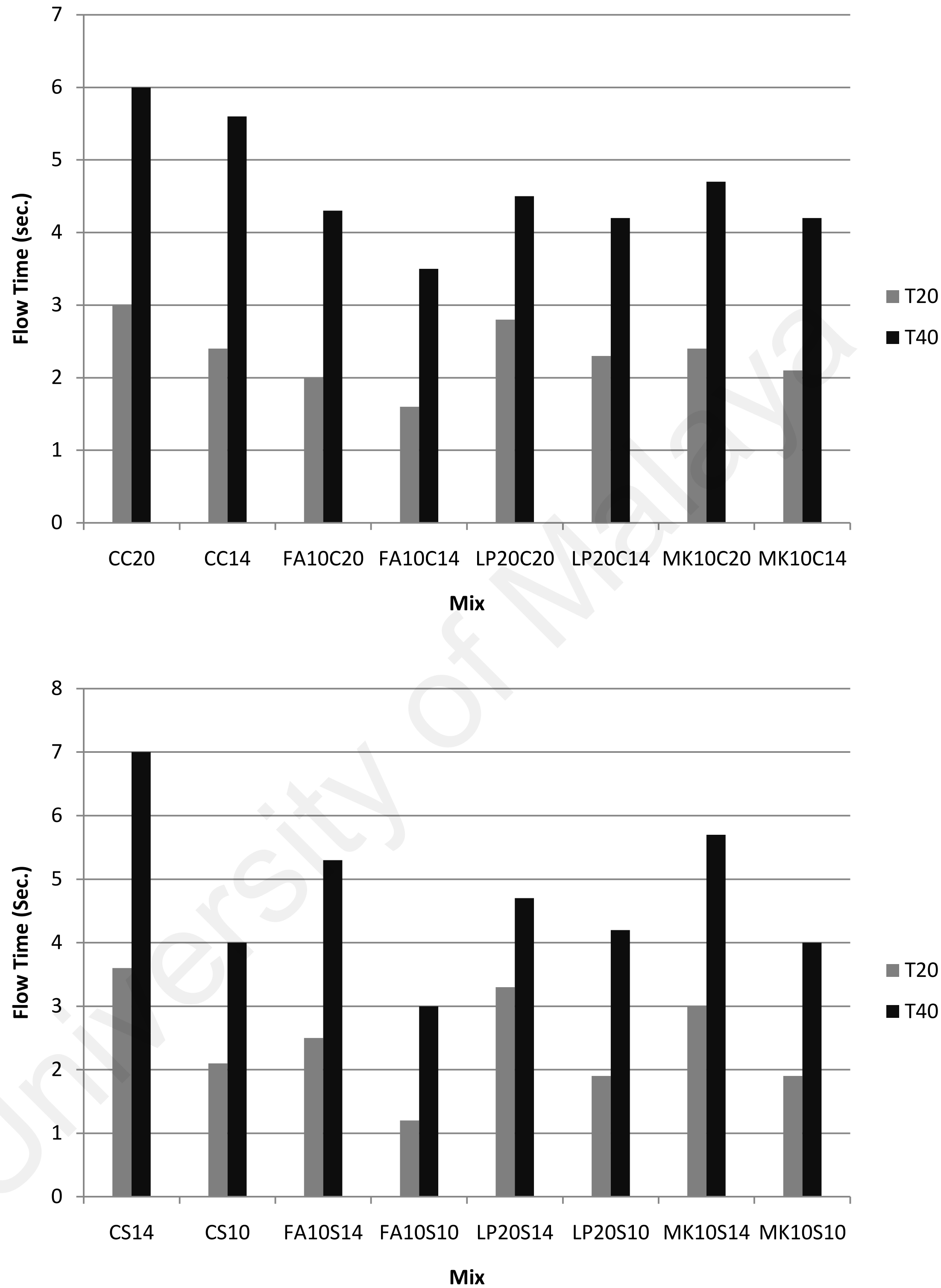


Figure 4.58: T20 & T40 of different maximum size aggregate mixes

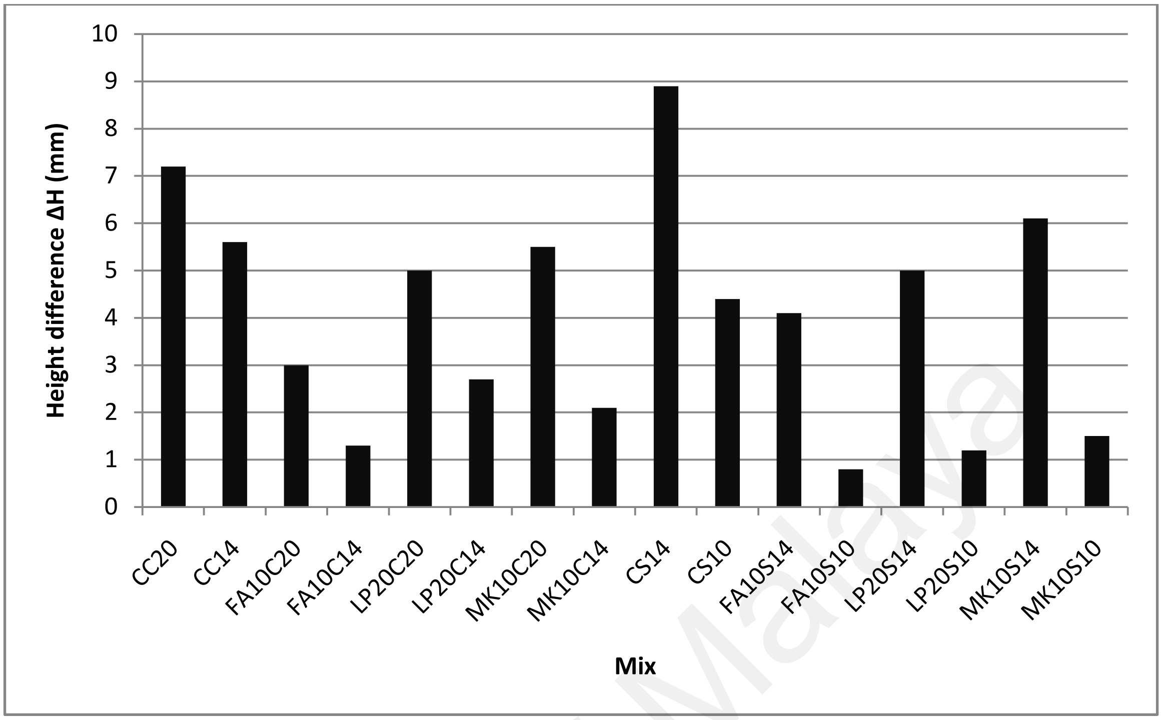


Figure 4.59: Height difference ΔH of J-ring test of different maximum size mixes

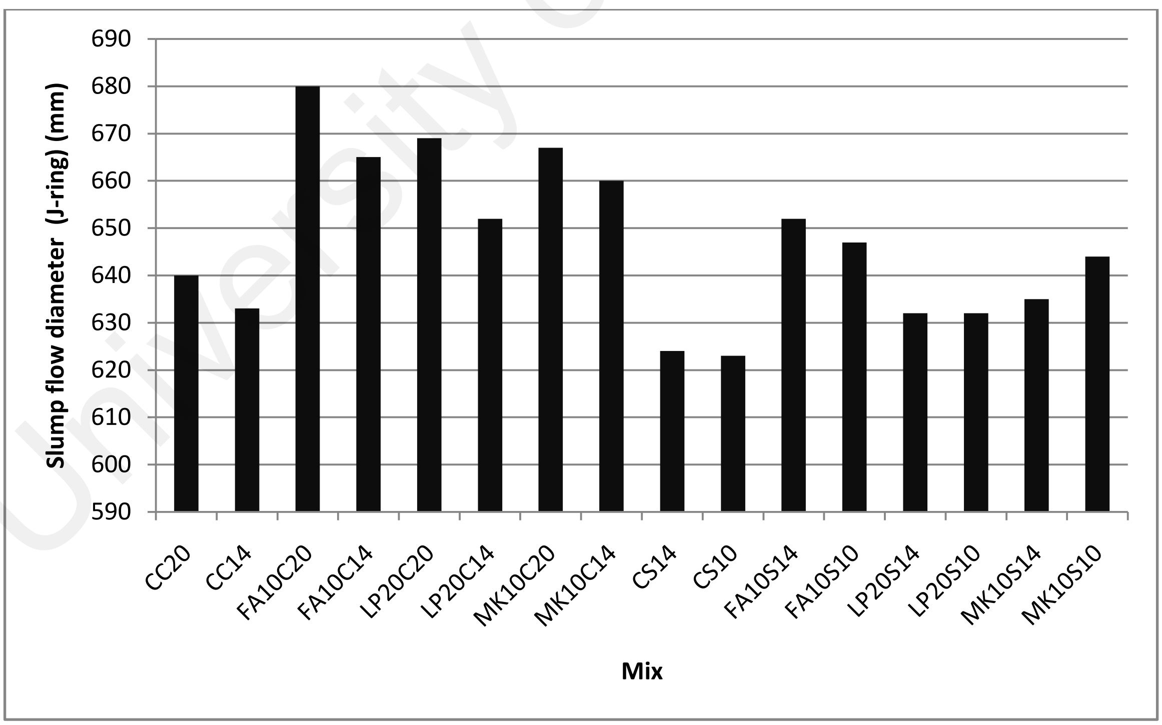


Figure 4.60: Slump flow diameter of different maximum size mixes

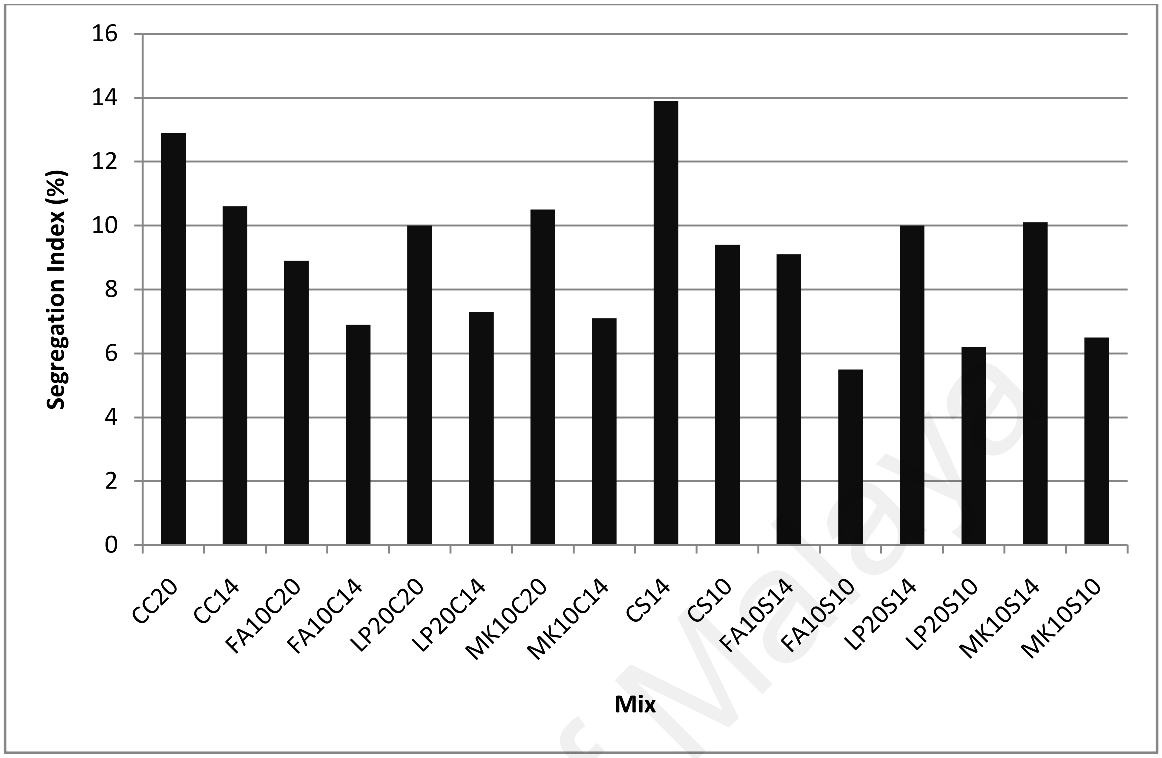


Figure 4.61: Segregation index of different maximum size mixes

4.6.4 Effect of coarse aggregate grading

The effect of grading of aggregate on slump flow diameter ranges between 645 to 670mm as shown in Figure 4.62. These values clearly show that the increment occurs in slump flow diameter of graded aggregate mixes when compared to slump flow diameter of gap grade and single size aggregate mixes. Graded aggregate mixes have high packing density i.e. low voids ratio when compared with other mixes. For the same mortar content, the excess mortar required to encapsulate and disperse the aggregate particles of graded aggregate mixes is less compared to the other types of grading aggregate mixes. Therefore, the flow ability of graded aggregate mixes is better. This is supported by many previous studies namely (Golterman et al., 1997, Glavind et al., 1993, Johansen and Andersen, 1989, Johansson, 1979, and Wong & Kwan, 2002)

For the same mix proportion, single size aggregate in concrete increases yield stress and viscosity when compared with graded aggregate mixes (Hu & Wang, 2011). In contrast, EFNARC (2002) reports that when compared to continuous grade aggregate, the use of gap graded aggregate in concrete improves flow ability by reducing the internal friction between particles.

In Figure 4.63, the T_{50cm} results vary depending on the type of grading. Indeed, these results range between 1.8 to 3.6 seconds. For the same mix proportion, the T_{50cm} of the gap grade aggregate mixes on the table after lifting the cone is higher than the T_{50cm} of the graded aggregate mixes. The increased T_{50cm} of gap grade aggregate mixes is due to the fact that the mortar layer thickness around aggregate particles is less than that in graded aggregate particles because the mortar acts as a lubricant between aggregate particles.

Figure 4.64 shows the effect of grading on the values of T_f and T_{f5min}. The mixes which contain 20mm graded aggregate exhibit lower values of T_f and T_{f5min} than those which contain 20mm gap graded aggregate. In addition, Figure 4.64 indicates their mixes which contain 14mm graded aggregate exhibit lower values of T_f and T_{f5min} than those containing 14mm single size aggregate.

It is clear that the difference between the T_f values of 20mm graded and gap grade mixes is less than the difference between the T_f values of 14 mm graded and single size aggregate mixes. This is attributed to the fact that single size aggregates have more large aggregate particles relative to graded aggregate with maximum size 14mm than gap grade aggregate particles relative to graded aggregate with maximum size 20mm. The orifice of the V-funnel box impedes the passing of particles as it is larger and in turn delays the speed of discharge of fresh concrete. Khaleel (2011) indicated that the presence of 20mm single sized aggregate in SCC mixes lead to increase in T_f through V-funnel orifice compared to 20mm well graded aggregate. Al- jabri (2005) showed in his study that the use of gap graded aggregate would delay the time of concrete flow through V-funnel orifice due to increase in the number of blockage particles close to the orifice.

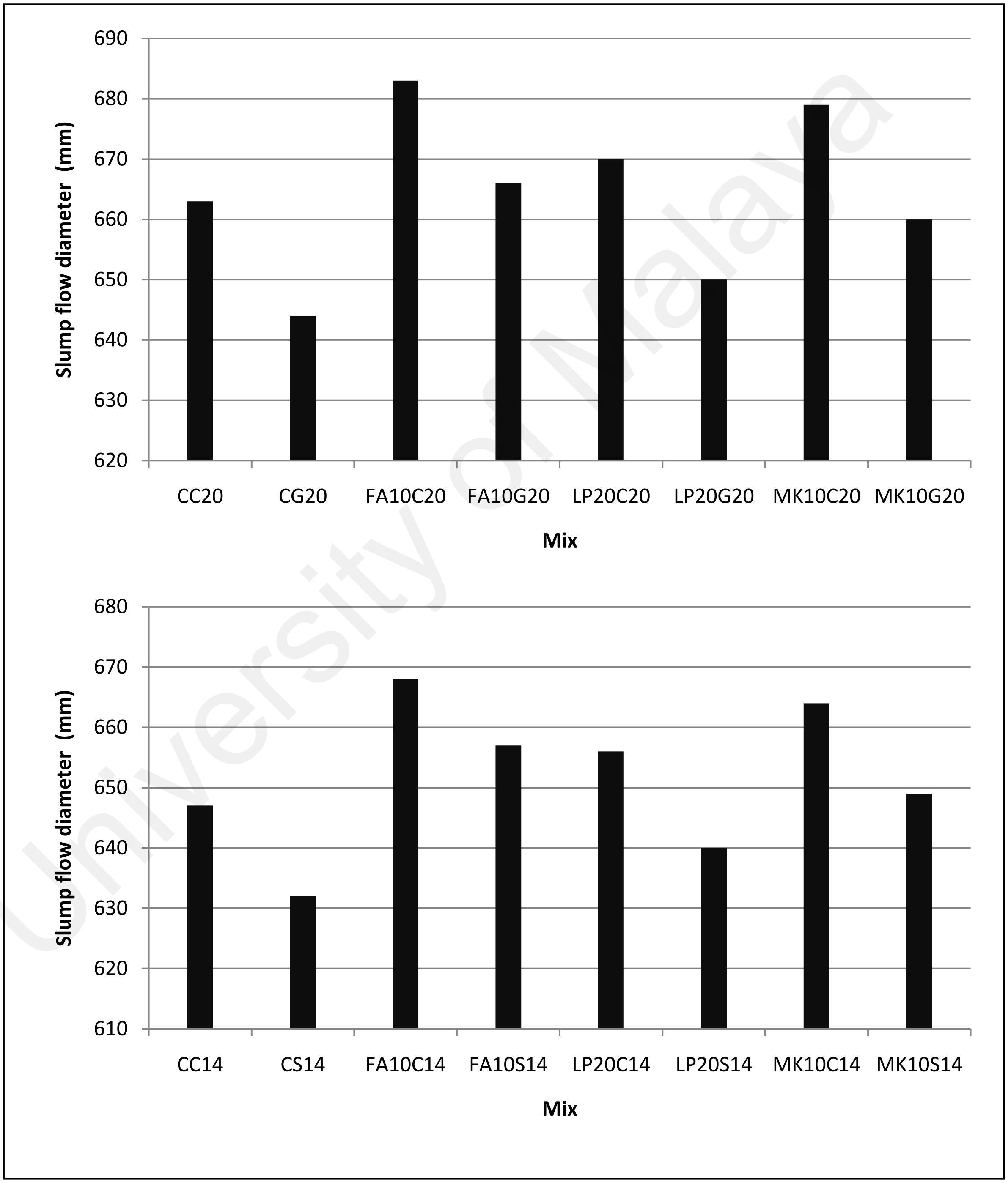


Figure 4.62: Slump flow diameter of different grade of mixes

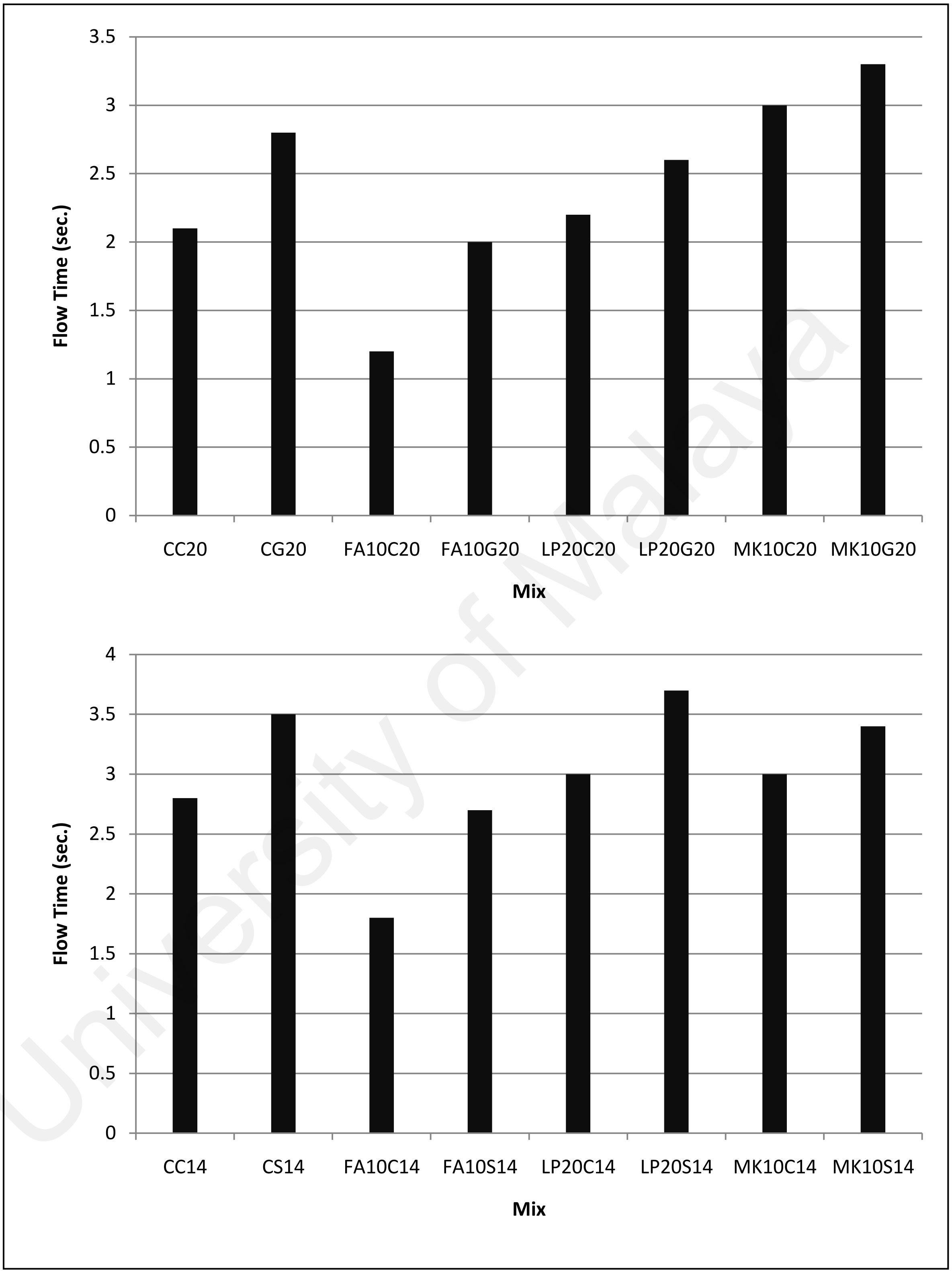


Figure 4.63: Flow time at spread of 50cm of different aggregate grades (graded, gap and single) of mixes

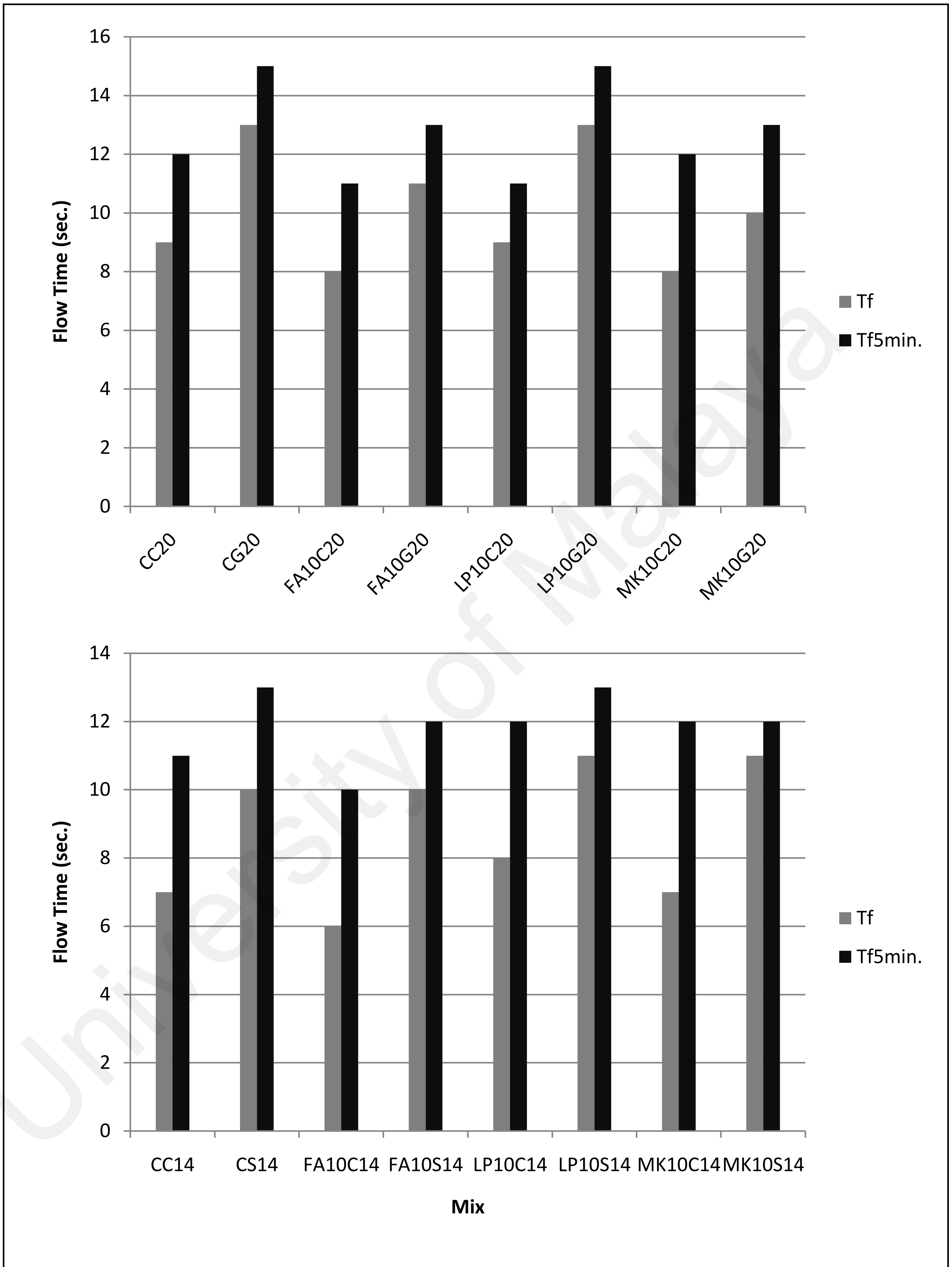


Figure 4.64: Flow time through V-funnel of different aggregate grades of mixes

The effect of aggregate grading in concrete on blocking ratio and T_{20cm} as well as T_{40cm} values is shown in Figure 4.65 and 4.66 respectively. Figure 4.65 indicates that blocking ratio values increase with the use of graded aggregate in concrete mixes when compared with using gap grade and single sized aggregate mixes. In addition to this, T_{20cm} and T_{40cm} values are reduced with the use of graded aggregate in concrete mixes when compared with using gap graded and single size aggregate in concrete mixes as shown in Figure 4.66. The graded aggregate mixes exhibit lower tendency for confinement behind steel bars of the L-box. Therefore, the percentage of the concrete passing through and around the steel bars of the L-box is more than the concrete mixes which contained gap graded and single sized aggregate. This was also observed by Koehler and Fowler (2007).

The results of ΔH for studying the effect of aggregate grading are illustrated in Figure 4.67. From this figure it is evident that the use of graded aggregate in concrete gives less value of ΔH than the use of gap graded and single sized aggregate in concrete. Indeed, the concrete which contained 20mm graded aggregate gives less ΔH values than those which contained 20mm gap graded aggregate. Moreover, the mixes which contained 14mm graded aggregate led to a decrease in the values of ΔH when compared with the mixes which contained 14mm single sized aggregate. This may be attributed to the fact that the use of graded aggregate in concrete allows the concrete mix to pass easily between the steel bars more than the use of gap grade and single sized aggregate in concrete. Figure 4.68 indicates the slump flow diameter (SFD) values from J-ring test of the mixes with graded and gap graded aggregates. The use of gap graded aggregate in the mixes leads to a reduction in the SFD of J-ring test. In addition, Figure 4.68 shows the slump flow diameter values from J-ring of single size and graded aggregate mixes, whereby the presence of single size aggregate in the concrete reduces the slump flow diameter from J-ring compared to graded aggregate.

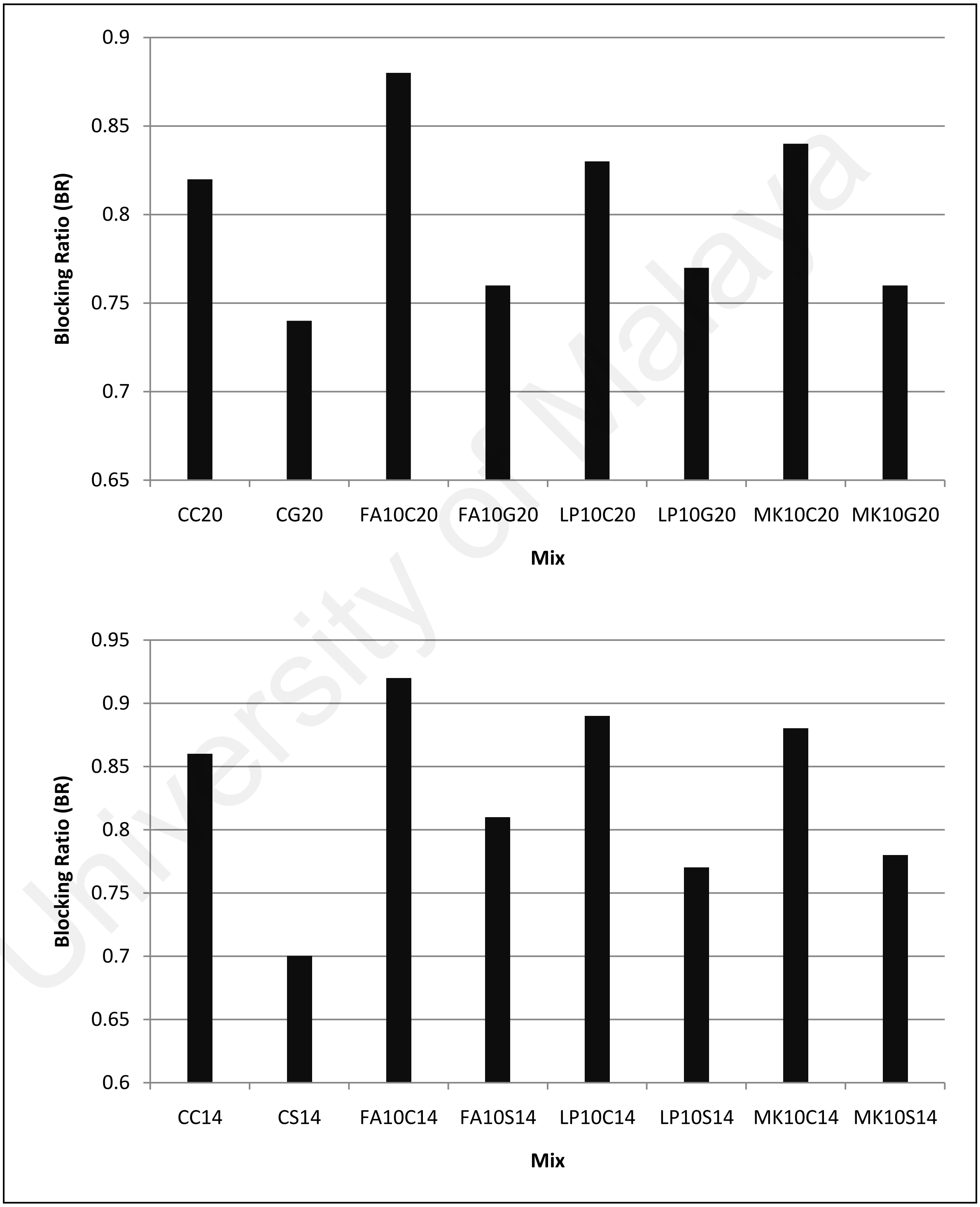


Figure 4.65: BR of different aggregate grades (graded and single) of mixes

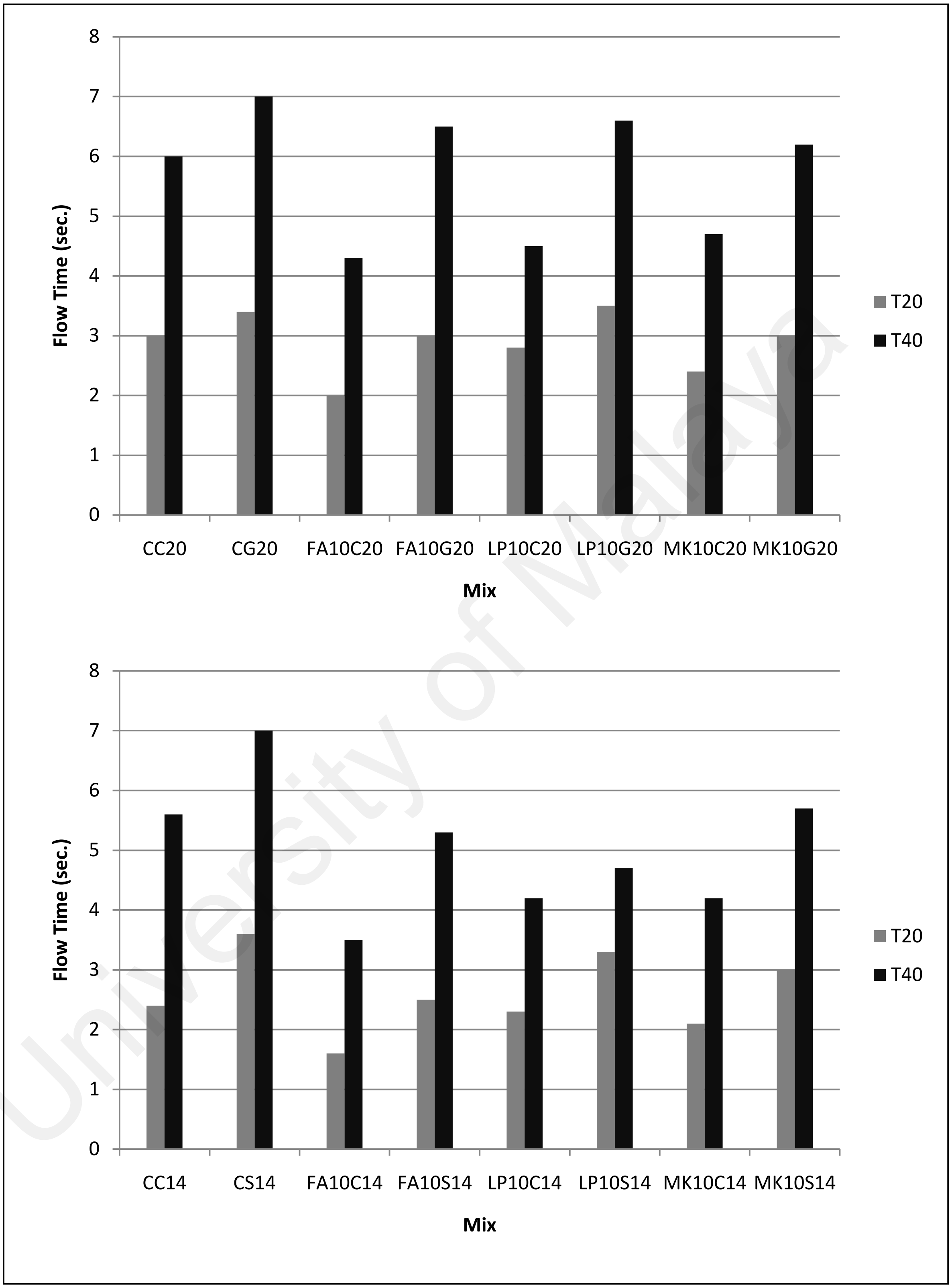


Figure 4.66: T20 & T40 of different aggregate grades (graded and single) of mixes

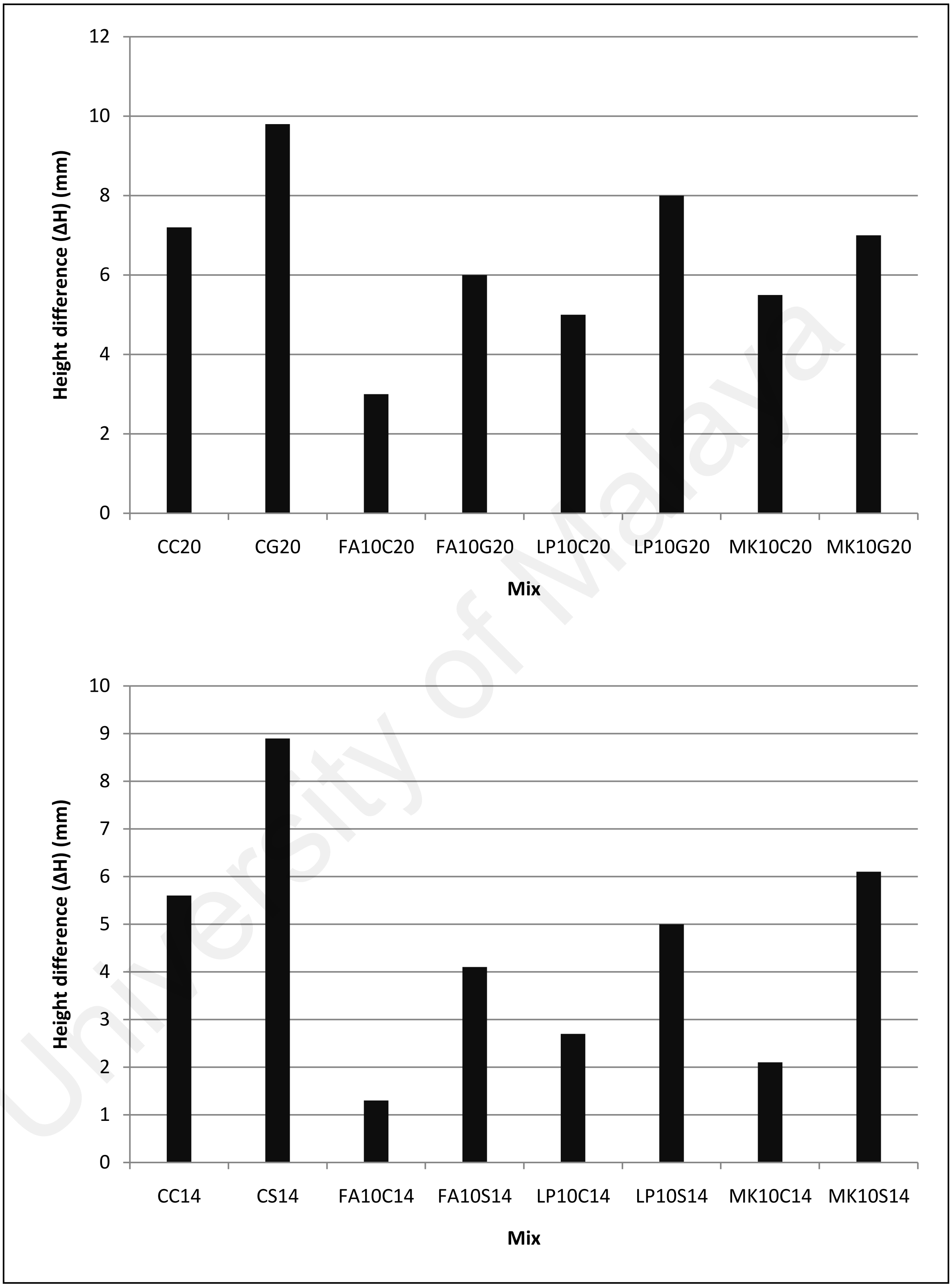


Figure 4.67: Height difference of different aggregate grades (graded and single) of mixes

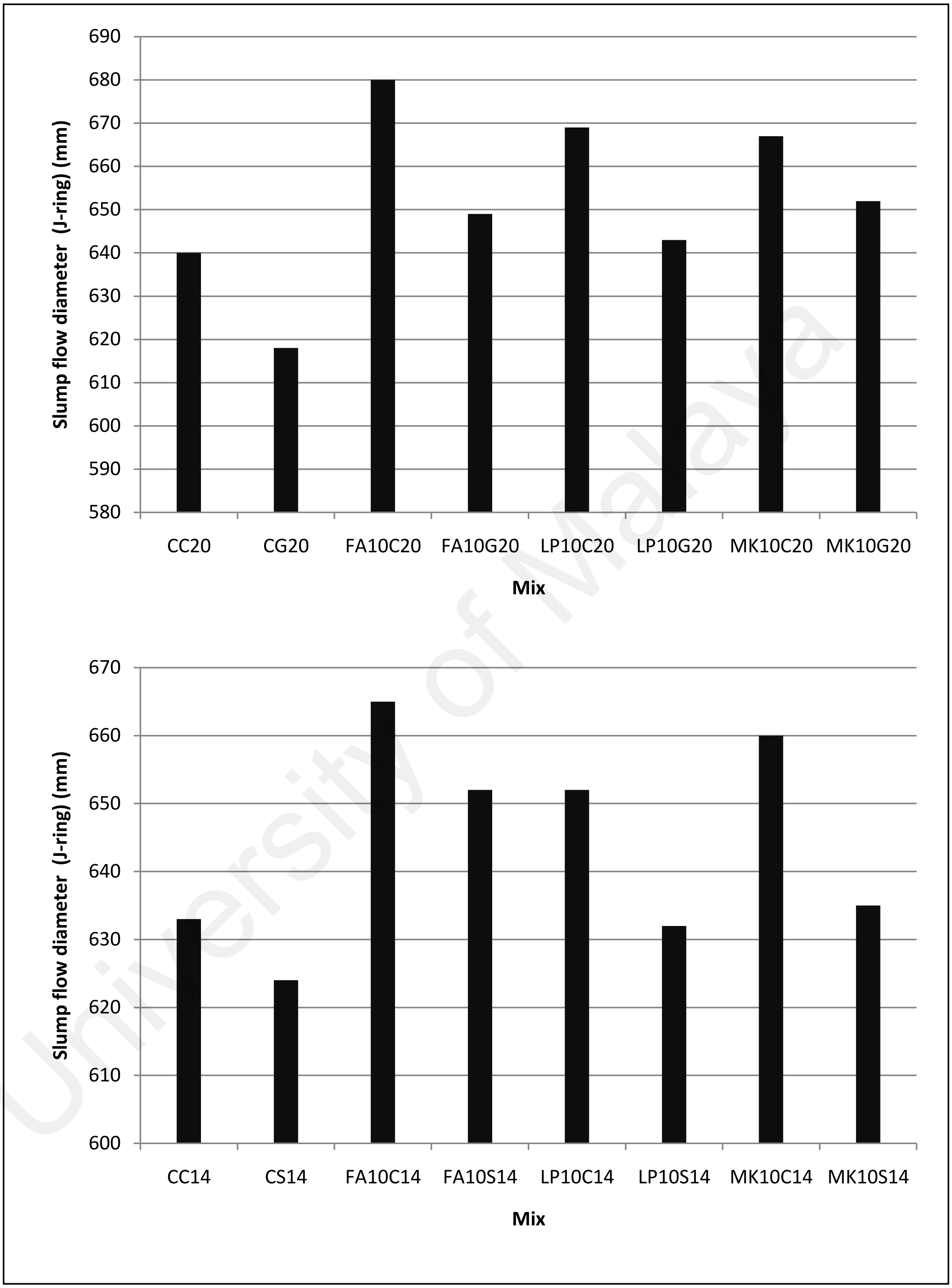


Figure 4.68: Different height of different aggregate grades (graded and single) of mixes

Figures 4.69 indicate the diversity in segregation index values due to the effect of grading of coarse aggregate in the mixes. Figure 4.69 appears to show that the mixes which contain graded aggregate are less prone to segregation than the mixes which contained gap graded and single sized aggregate. This is clearly evident from the segregation index data. Indeed, segregation index values reduce when graded aggregate was used in the mixes compared with other types of grading of coarse aggregate. This is due to the fact that graded aggregate has more excess mortar to cover aggregate particles. In addition to this, the flow of particles of graded aggregate in the mixes is more homogenous and therefore, the segregation tendency would be less (Xie et al., 2005).

Rached et al. (2009) studied the effect of grade of coarse aggregate on the workability of concrete with different sand to coarse aggregate ratios. This study showed that gap grade of coarse aggregate is coarser than continuous grade, thus concrete mix could bleed.

Berke et al. (2002) reported that the mixes with poorly graded aggregate need additional viscosity modifying admixture to enhance the stability of fresh self compacting concrete mixes and this was supported by Bury and Christensen (2002)

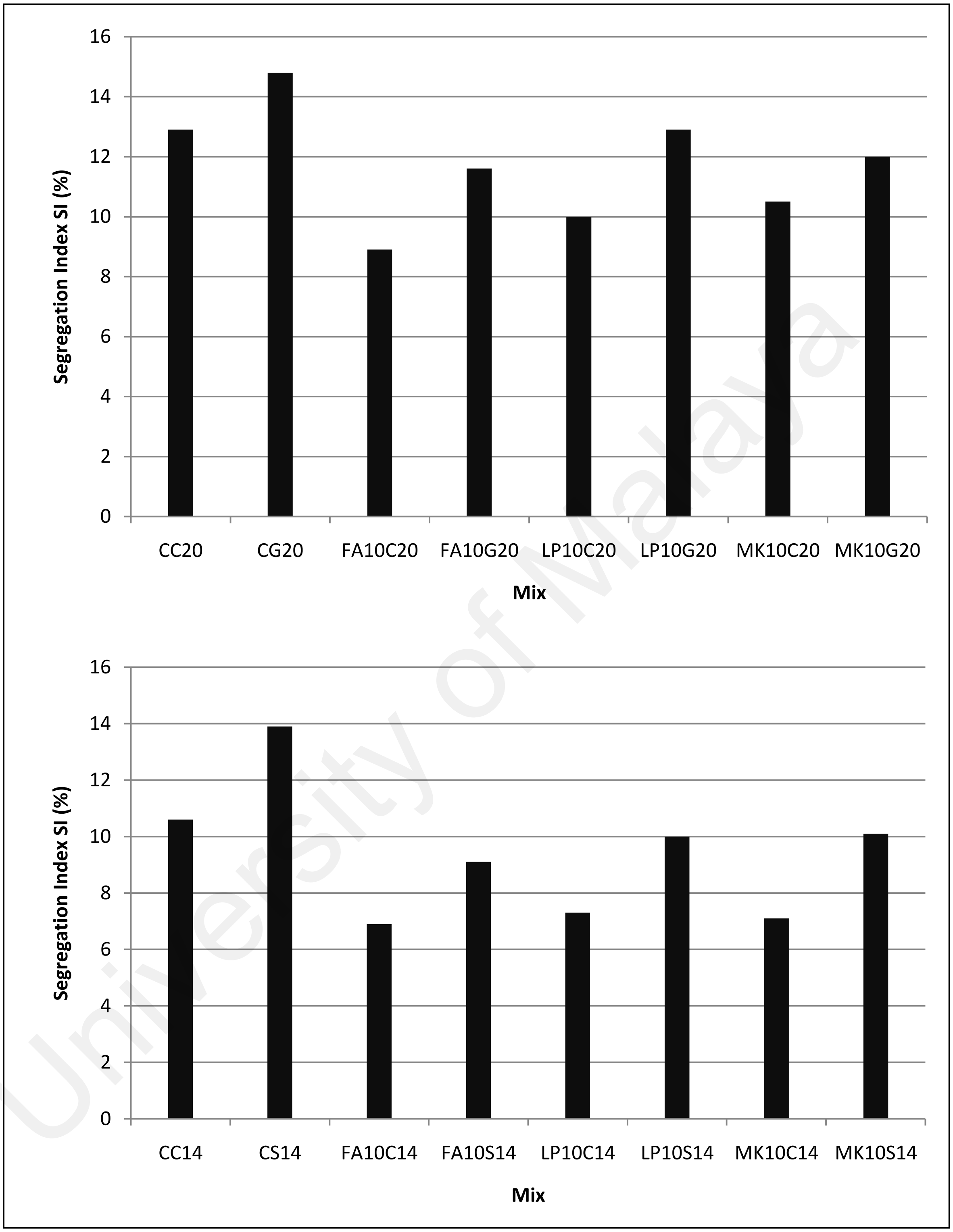


Figure 4.69: Segregation index of graded and single size aggregate concrete mixes

4.6.5 Effect of coarse aggregate flakiness

It is apparent from Figure 4.70 that high flakiness ratio in the mixes of 10mm aggregate leads to a reduction in the slump flow diameter of mixes. In addition to this, as the high ratio of flaky particles increases, the T_{50cm} of fresh concrete also increases as shown in Figure 4.71. However, the rate of discharge of fresh concrete through the V-funnel opening increases as the flakiness ratio decreases as shown in Figure 4.72. The presence of a high ratio of flaky particles in concrete requires more mortar to flow easily through the opening of the V-funnel box as it is not easy for the particles to roll over each other. In a study conducted by Shilstone (1990), flaky particles significantly affect the flow ability of fresh mixes and makes the mixes prone to harshness. Chen et al. (2005) concluded that the mixes containing high flaky aggregates are less resistant to shear deformation.

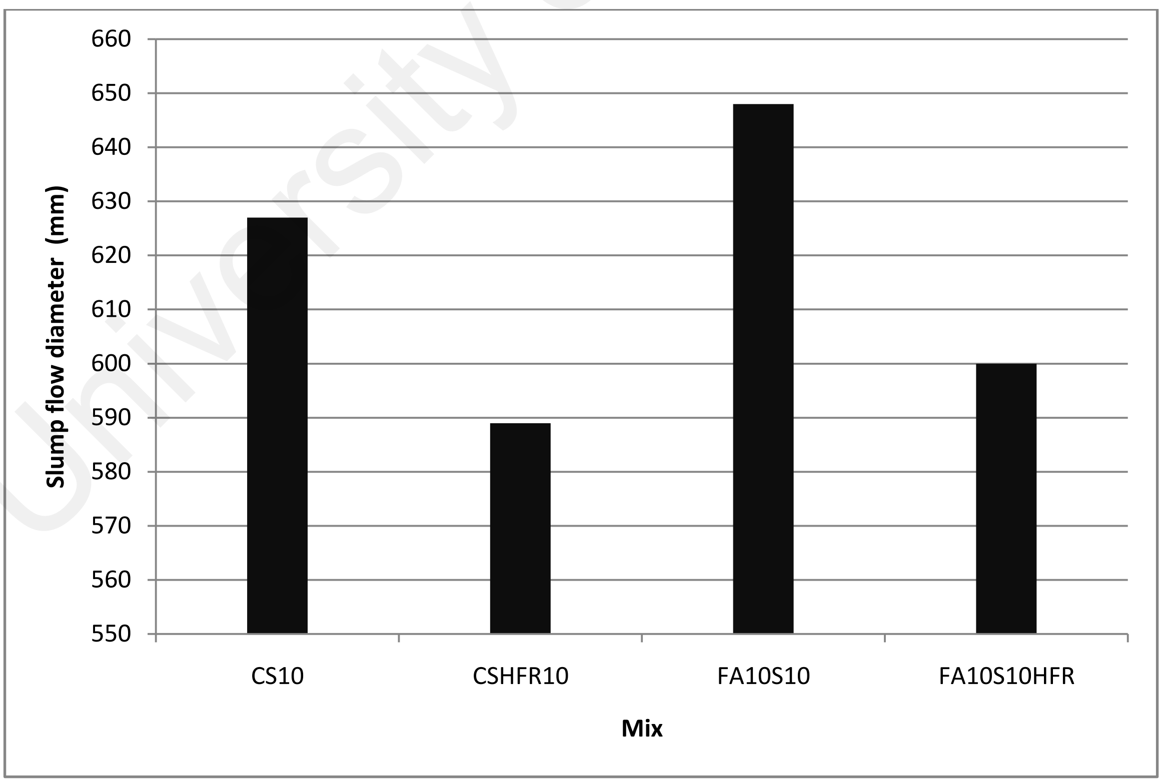


Figure 4.70: Slump flow diameter of different shape of aggregate concrete mixes

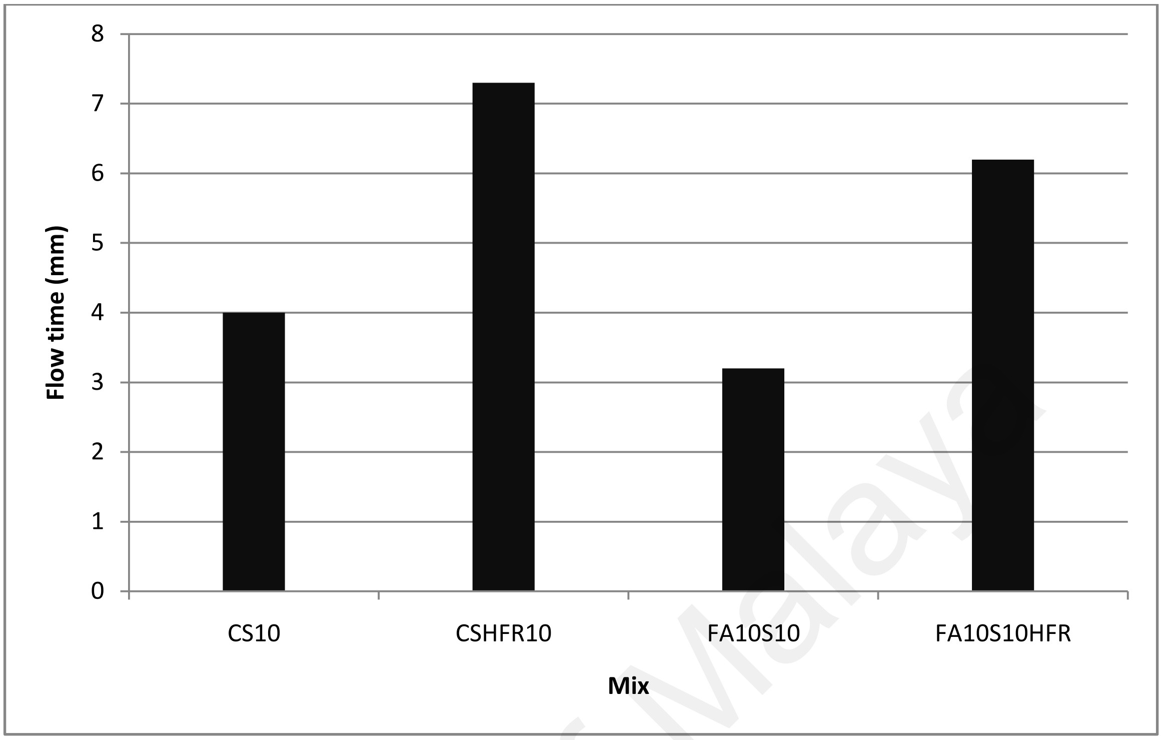


Figure 4.71: Flow time at 50cm of different shape of aggregate concrete mixes

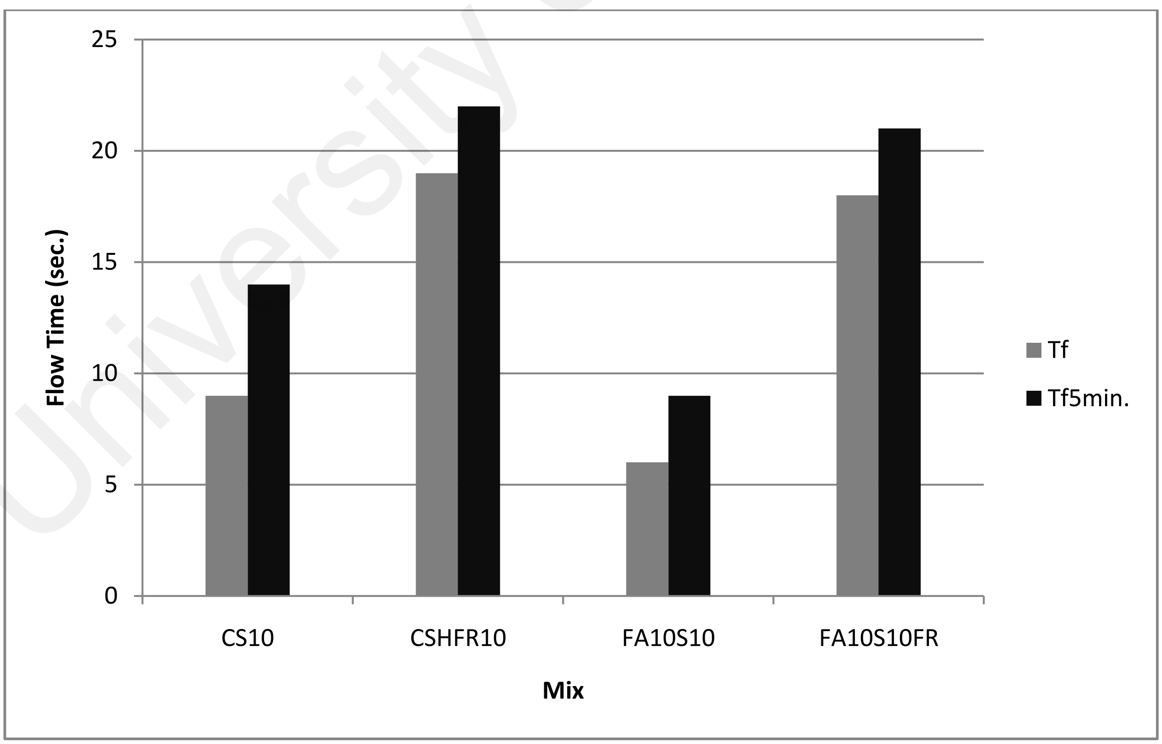


Figure 4.72: Flow time V-funnel of different shape of aggregate concrete mixes

The results of blocking ratio vary based on the flakiness ratio in aggregate as shown in Figure 4.73. Indeed, it shows that as the flakiness ratio of aggregate increases the percentage of concrete passing through steel bars is reduced. This is clear from blocking ratio results, where the results of blocking ratio are low when using a high ratio of flaky particles in the mixes. This may be due to the fact that high flakiness ratio in the mixes leads to interlocking of the particles and thus causing blocking. In addition, Figure 4.74 illustrates the results of the flow time for the L-box at 20 and 40 cm distance. T_{20cm} and T_{40cm} values increase with the use of aggregate with a high amount of flaky particles.

From the results shown in Figures 4.75 and 4.76, it is evident that the high amount of flaky particles increases the difference in height and reduces the slump flow diameter of the J-ring of fresh concrete through steel bar spaces so that the difference in height increases with the increase in flaky particles in concrete. This may be due to the fact that concrete with high amounts of flaky particles is more likely to be confined behind reinforced bars which in turn reduce the amount of passed concrete. This agrees with the study conducted by Roussel (2009). Santhanam and Amal Raj (2004) conducted study about the effect of flakiness ratio in coarse aggregate on some fresh properties of SCC and found that increase in flaky particles in coarse aggregate led to increase in the difference in height in U-box test. This is due to the occurred blockage of particles at steel bar which in turn led to reduce the passing ability of concrete.

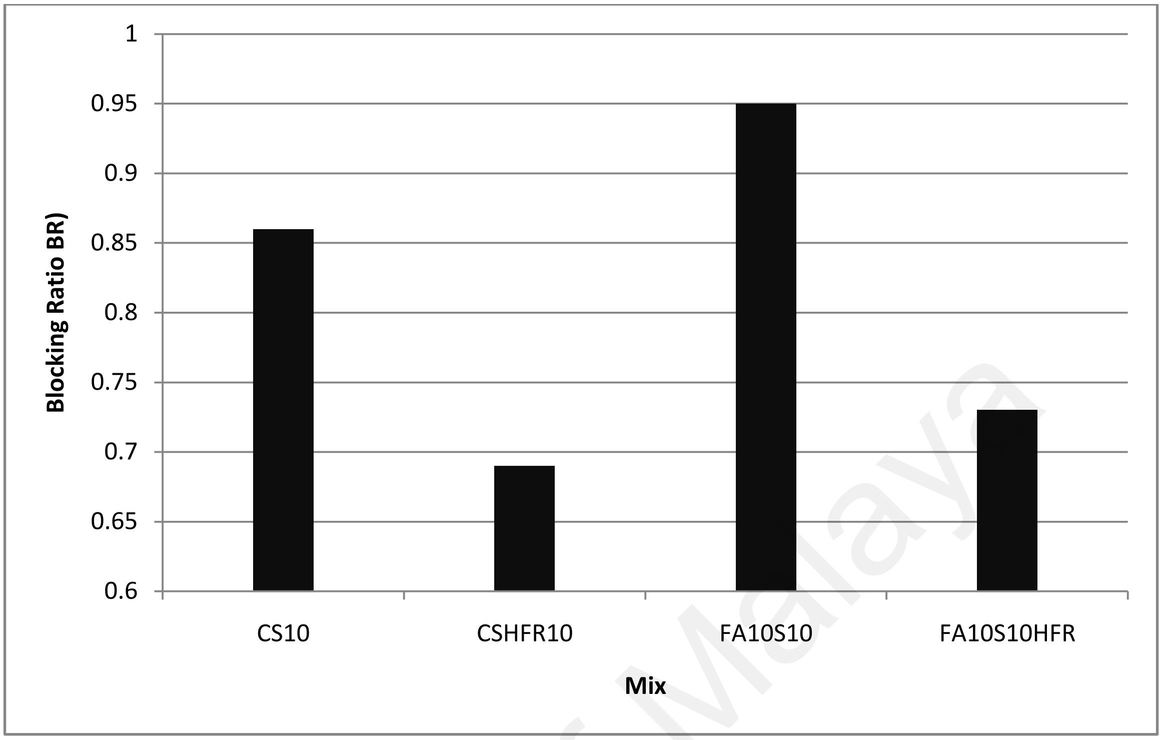


Figure 4.73: BR of different shape aggregate concrete mixes

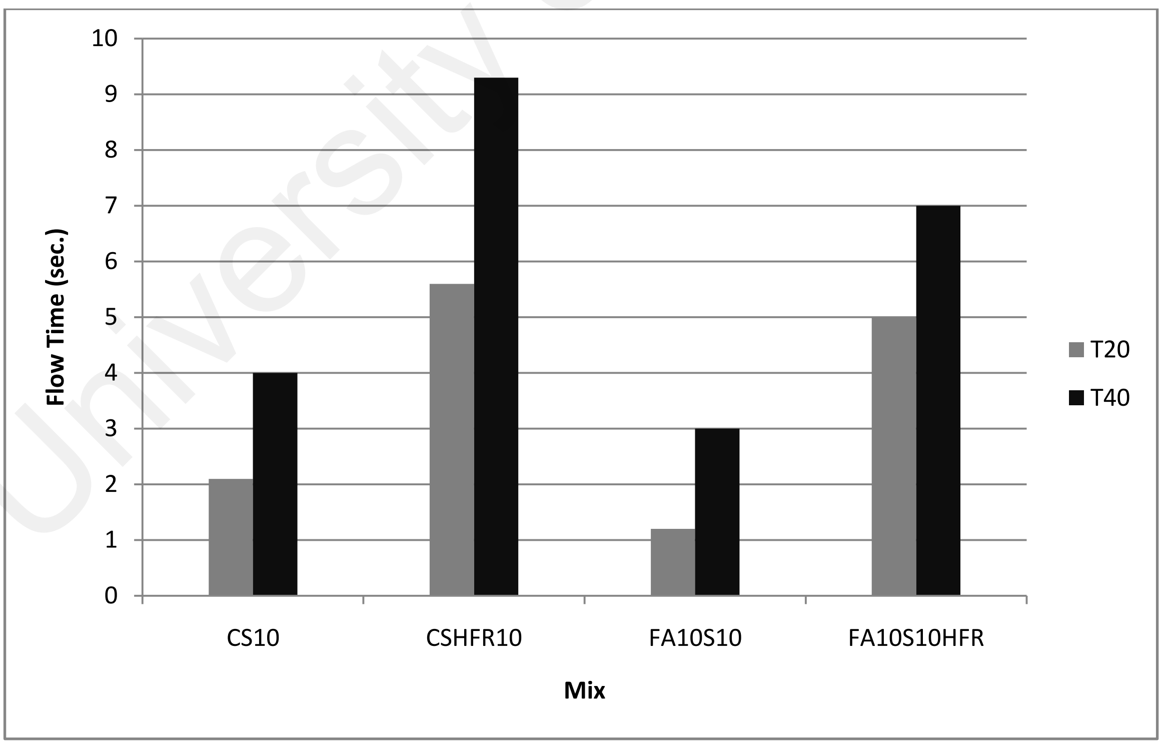


Figure 4.74: T20 & T40cm of different shape aggregate concrete mixes

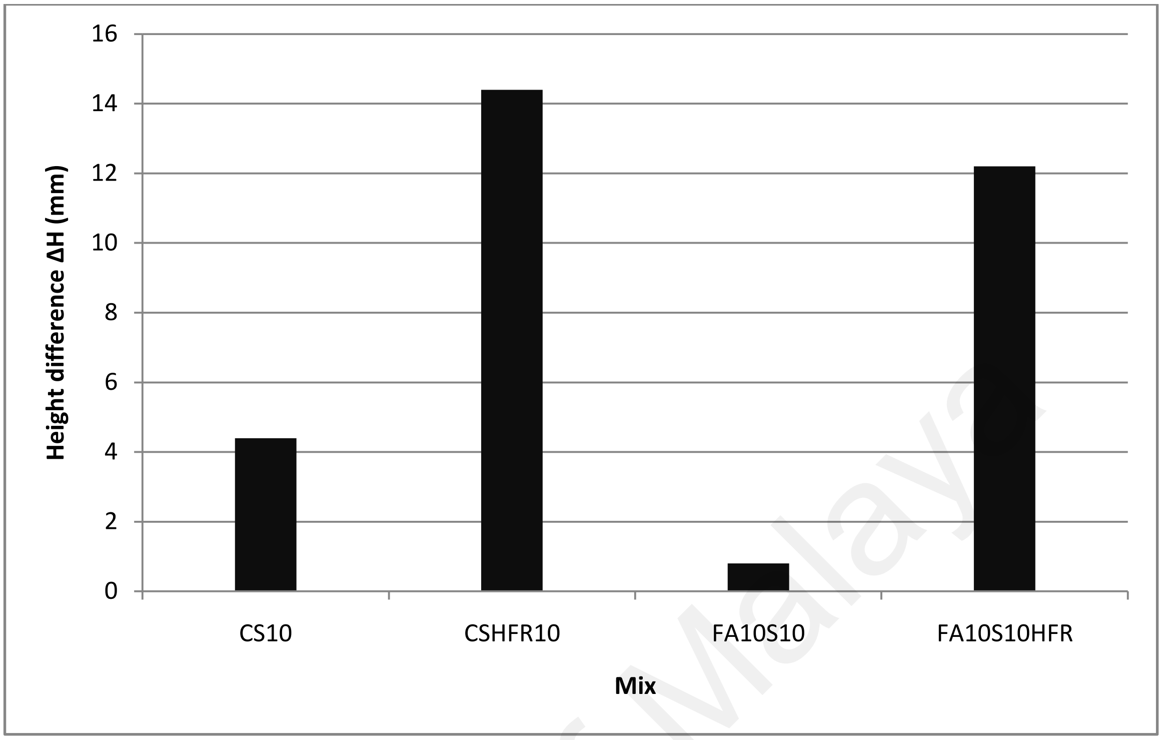


Figure 4.75: Different height of different shape aggregate concrete mixes

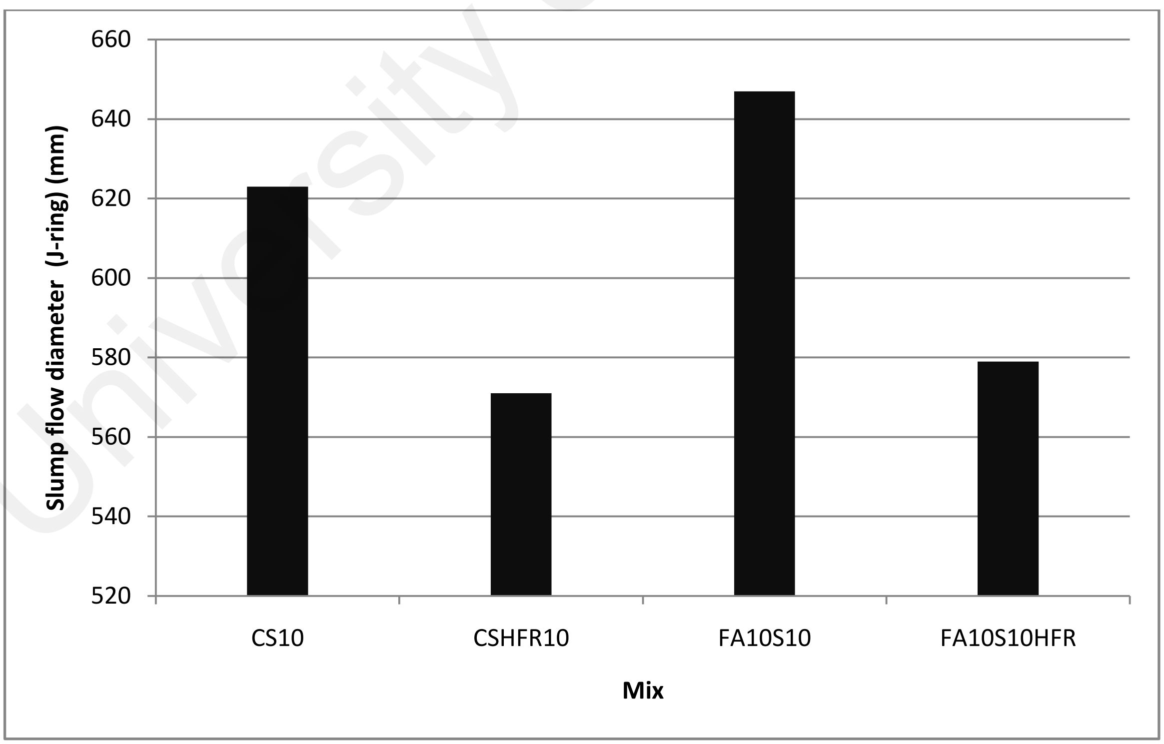


Figure 4.76: Slump flow diameter of J-ring of different shape aggregate concrete mixes

Figure 4.77 shows the effect of flakiness ratio on the stability of concrete mixes. The existence of a high ratio of flaky particles makes it more prone to segregation. Indeed, segregation index values increase with the increment in flakiness ratio of coarse aggregate. This is in agreement with Shilstone (1990). A high ratio of flaky particles in concrete increase voids in the mixes and thus the required excess mortar around particles is less. Siswosoebrotho et al. (2005) concluded that the increased flaky content of coarse aggregate reduces of the stability of asphalt concrete.

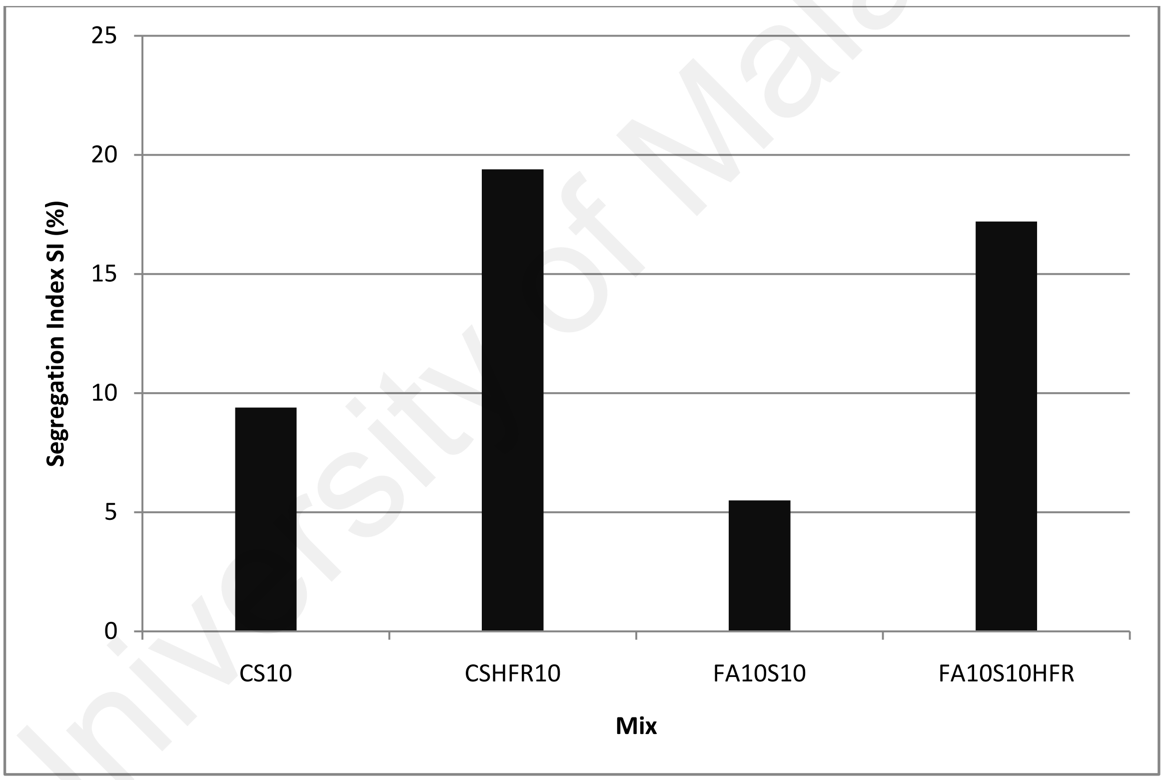


Figure 4.77: Segregation index of different shape aggregate concretes

4.7 Hardened properties of concrete

The hardened concrete test results of all mixes are presented and discussed in this section. The effects of type of pozzolan and filler powders, different coarse aggregate properties i.e. maximum size, grading and shape on the compressive, splitting tensile and flexural tensile strengths as well as drying shrinkage and water absorption are presented and discussed.

The results shown in Figure 4.78 are for hardened density of different mineral admixture mixes at 28 day. The range of density values varies between 2330 to 2465 kg/m^3. The results reveal that the fly ash mixes have lower density compared with other mixes due to the low specific gravity of fly ash which leads to increase in the volume of the mix and thus reduces aggregate content for balancing of mix volume. The difference in density between metakaolin mixes with control mixes is small despite the lower specific gravity of metakaolin. This is due to the pozzolanic activity of metakaolin mixes whereby the pozzolanic reactions with cement hydration products lead to a consumption of the $Ca(OH)_2$ crystals and the production of CSH. This increases the density of paste structure by filling the capillary pores. However, the highest density is the density of limestone mixes due to the high specific gravity of limestone powder as well as the filling effect of limestone powder which fills the voids between cement particles. In addition to this, the acceleration effect of limestone powder accelerates the cement hydration processes and thus improves the density. The effect of aggregate properties on the density is small. Indeed, the presence of small size aggregate gives higher density. Moreover, the use of graded aggregate leads to higher density compared to the other grading whilst the use of aggregate with high amounts of flaky particles leads to a reduction in density as shown in Figure 4.78.

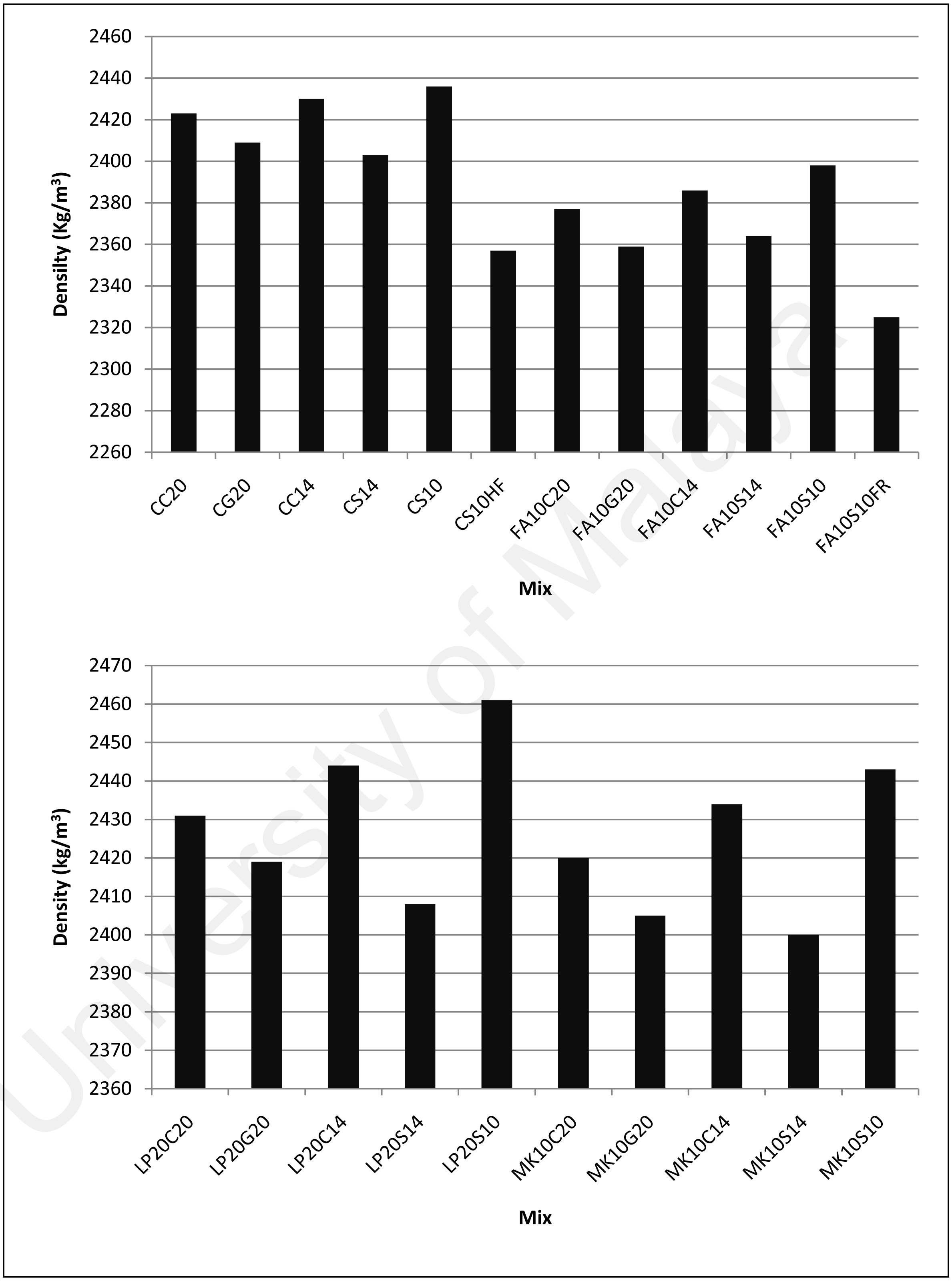


Figure 4.78: Hardened densities of all concrete mixes

4.7.1 Effect of mineral admixtures

The compressive strength results of all mixes at the same mix proportions are listed in Table 4.7.The strength development for control, filler and pozzolanic mixes up to 180 days is shown in Figures 4.79. It can be observed that limestone mixes enhance the strength in the long term more than the control and pozzolanic mixes as shown in Figure 4.79.

This is due to the combined effect of physical and chemical properties of limestone as mentioned in section 4.4.3. After 1 day limestone mixes exhibit a higher relative strength than after 28 and 180 days. In addition, as the age increases, the relative strength decreases. For example, after 1 day, the relative strength of LP20C20 is 1.67 but as age increases and especially at 28 days the relative strength became 1.52 and 1.32 at 180 days as shown in Figure 4.80. This agrees with Nehdi (1998), Felekoğlu et al. (2006),.

Guemmadi et al. (2009) indicated in their study that the concrete mixes which contained limestone filler gave high primary strength when compared with the mixes without mineral admixtures. This behaviour results from the fact that the limestone powder accelerates the formation process of calcium carboaluminate hydrate which could be influential on the hydration rate. In the long term, the contribution of limestone in improving the strength becomes minimal.

The pozzolanic effect of fly ash and metakaolin on compressive strength is apparent from the results and is shown in Figure 4.79. After 1 day, the relative strength of metakaolin mixes is low as when compared with the relative strength of 28 days as shown in Figure 4.80. This is in agreement with Al-Jabri (2005). This is due to high dosage of superplasticizer of metakaolin mixes which may delay the reaction of cement hydration and thus the setting action. In addition, the pozzolanic activity at the early ages is small. After 28 days the percentage increase is higher than after 1 and 180 days which could be due to the pozzolanic activity of metakaolin, as mentioned in 4.4.2.1. According to Fook (2004), with time the difference in compressive strength between metakaolin mix and control mix is smaller. This is mainly attributed to the fact that the reaction between cement hydration products and metakaolin at the later ages is slowed down with time due to the completion of the reaction processes in metakaolin mixes.

Wild et al. (1996) reported that there are some factors which improve the strength of metakaolin mixes. Metakaolin acts as filler as well as its role in accelerating the cement hydration. The filler effect of metakaolin improves the packing density of the paste structure while the pozzolanic activity contributes to improving the strength especially at the ages between 7 and 14 days.

The fly ash mixes also reveal a low difference in strength with control mixes at the later ages and for the same reason mentioned above.

Ismeik (2009) observed in his study that the employment of fly ash gave less strength than the control mix at early ages whilst the strength improved at the later ages after 90 days.

The strength of fly ash mixes at 1 day is slightly higher than metakaolin mixes which could be attributed to the fact that metakaolin mixes have a high dosage of superplasticizer which may delay the cement hydration; and this has been reported by Hassan et al. (2000). After 3 days, the compressive strength of fly ash mixes is lower than metakaolin mixes due to the fact that the particle size of fly ash is coarser than the particle size of metakaolin.

This means that the required surface area for pozzolanic reaction is lower and thus the strength would be less. In addition, Figure 4.79 also indicates that metakaolin mixes give strength higher than fly ash mixes and this is contrary to the mortar mixes. This could be due to the fact that the paste content in fly ash mixes is higher and aggregate content is less compared that in metakaolin mixes and thus the transition zone is less.

Figure 4.80 indicates the relative strength of pozzolan and filler mixes. It illustrates the high relative strength of limestone mixes followed by metakaolin mixes and finally fly ash mixes. All mixes give strength higher than control mixes but with different levels.

Holschemacher and Klug (2002) reported that the rate of strength enhancement with time of SCC was similar to that of normal concrete; however, the utilizing of limestone powder as filler could increase the early enhancement of strength.

Justice (2005) studied the effect of using metakaoin and silica fume as pozzolanic material on the properties of normal concrete in fresh and hardened states. This study showed that the maximum increase of compressive strength of metakaolin mixes was at 28 days compared to other ages. This study also showed that metakaolin mixes exhibited higher strength at early ages (1day) compared to control mixes due to the filler effect of metakaolin in the mixes.

Table 4.7: Compressive strength results of all mixes

Symbol	Compressive Strength (MPa)						
	Age (day)						
	1	3	7	28	56	90	180
CC20	30	38	51	62	70	76	82
CG20	26	34	46	59	63	67	76
CC14	35	43	56	65	74	81	87
CS14	31	35	51	61	65	72	80
CS10	39	41	60	71	79	83	91
CSHFR10	25	29	39	47	55	59	61
FA10C20	38	53	68	77	79	84	92
FA10G20	32	48	63	73	76	79	88
FA10C14	40	47	71	79	82	88	98
FA10S14	37	43	63	71	74	81	88
FA10S10	45	55	68	80	85	94	101
FA10S10HFR	30	39	51	59	68	73	77
LP20C20	50	78	92	94	99	103	108
LP20G20	43	75	86	89	96	98	102
LP20C14	53	77	96	102	108	111	115
LP20S14	49	78	89	95	100	104	107
LP20S10	59	85	96	105	111	114	119
MK10C20	37	61	76	82	88	94	98
MK10G20	31	55	69	78	81	87	90
MK10C14	41	65	79	87	91	97	102
MK10S14	35	57	70	79	84	89	95
MK10S10	36	66	80	89	95	99	107

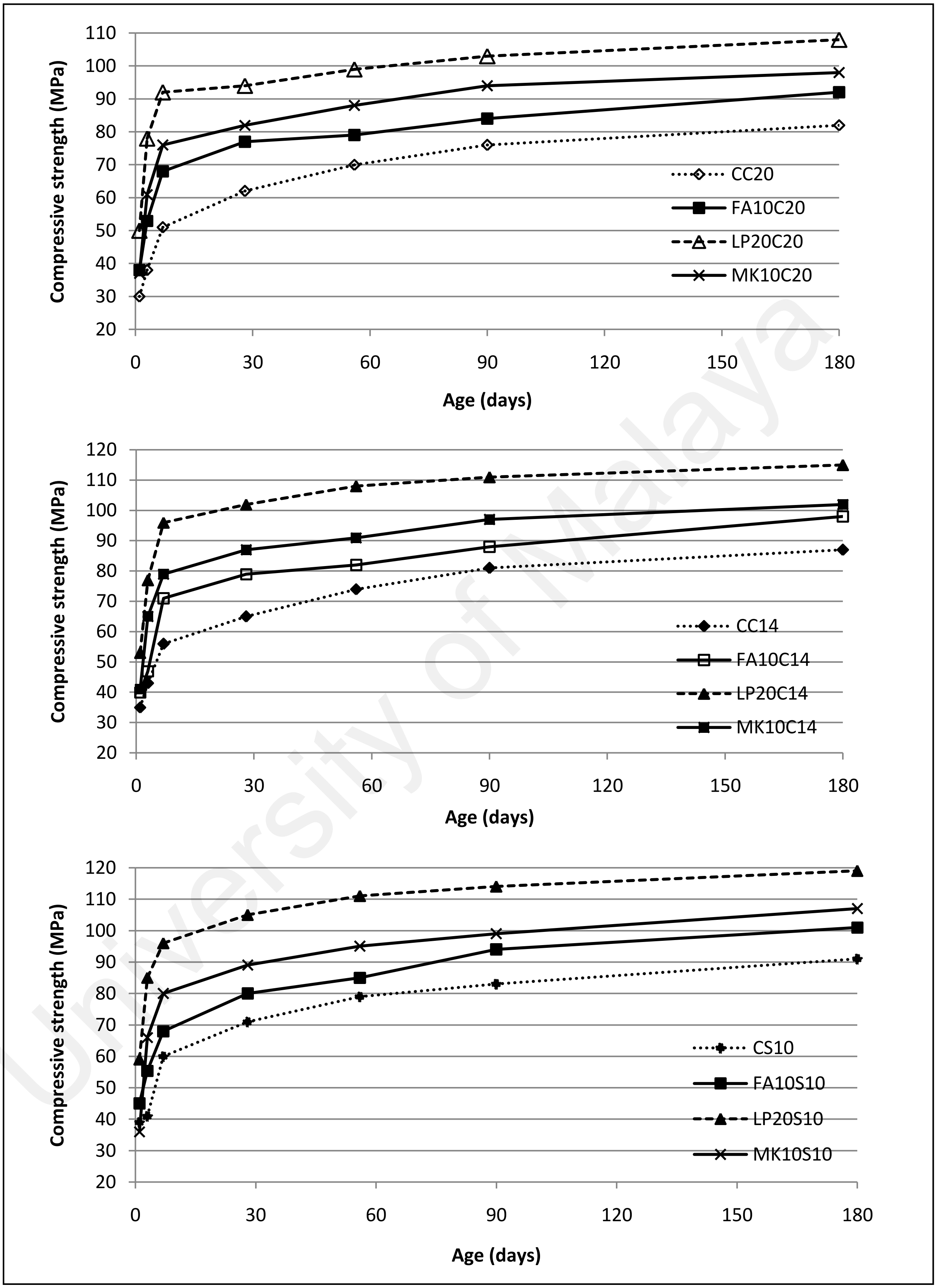


Figure 4.79: Compressive strength development for different mineral admixture mixes

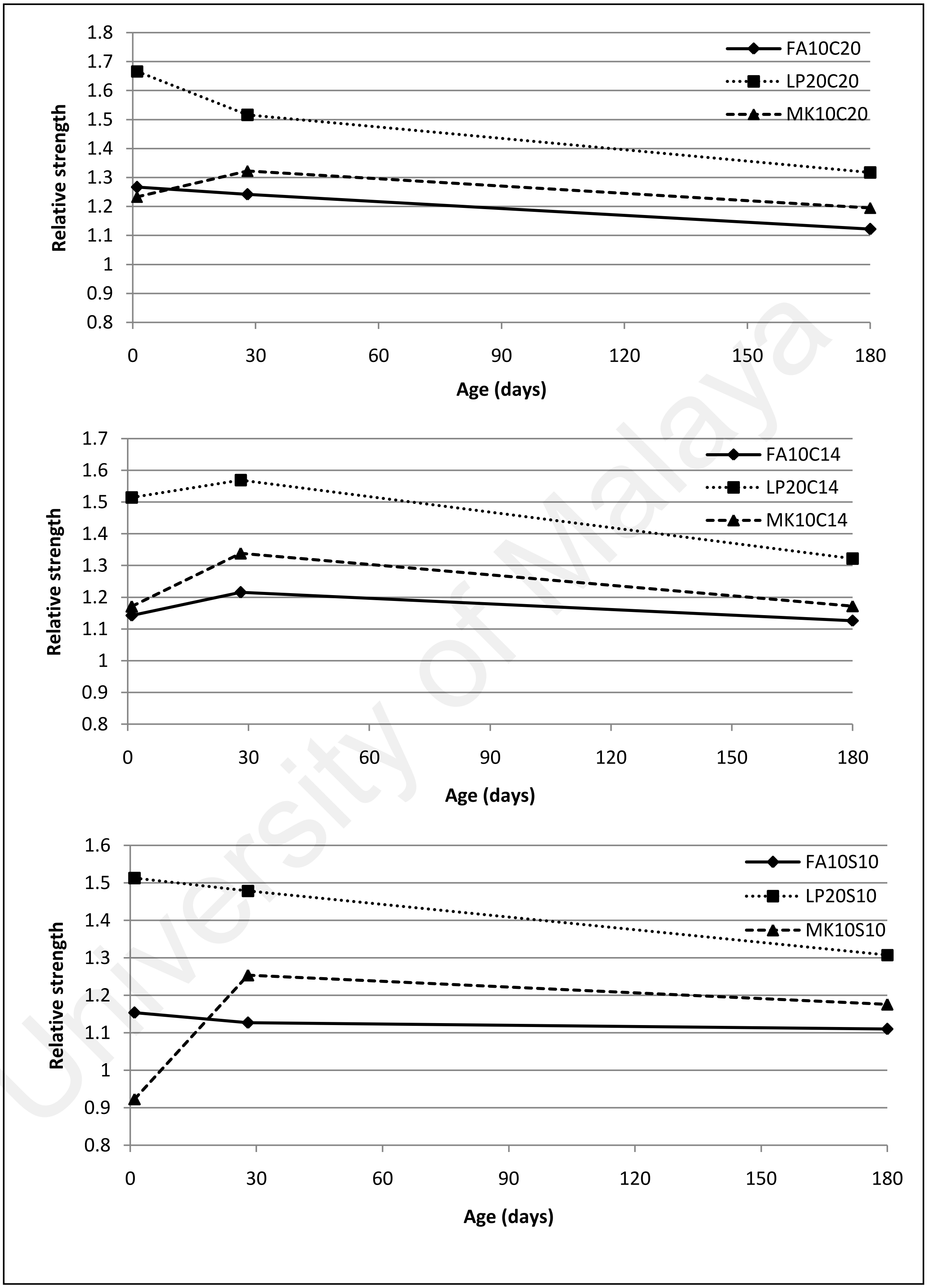


Figure 4.80: Relative strength of different mineral admixtures mixes

The results of splitting tensile strength for all mixes vary between 3.5 to 5.9 MPa as shown in Figure 4.81. It can be noted from Figure 4.81 that the tensile strength has been improved by the employment of limestone in the mixes. This enhancement in splitting tensile strength is due to the improvement which occurs in the interfacial transition zone by accelerating the formation process of calcium carboaluminate hydrate which is influential on the hydration of C_3S and hence improves the bond an interfacial transition zone. This improvement has a major affect on the splitting tensile strength as mentioned above. The splitting tensile to compressive strength ratios of all mixes are shown in Table 4.82. In limestone powder mixes, the splitting tensile strength to compressive strength ratios range between 5.8 to 6.0% at 28 days as shown in Figure 4.82. Avram et al. (1981) reported that splitting tensile to compressive strength ratios usually vary between 6 and 20 %. Generally, the splitting tensile to compressive strength ratio reduces as the strength and age increase (Mindess et al., 2003 cited in Habeeb, 2009) as also observed in the results of this study.

The employment of metakaolin and fly ash as pozzolanic material results in the improvement of the splitting tensile strength of concrete mixes when compared with control mixes. This is due to the enhancement which occurs in compressive strength as the pozzolanic activity of metakaolin and fly ash leads to an improvement in the bond strength of the transition zone. The improvement is due to the consume of $Ca(OH)_2$ in transition zone by its reaction with SiO_2 from the pozzolan to produce C-S-H. In metakaolin mixes, the tensile to compressive strength ratios range between 6.2 to 6.6% while in fly ash mixes vary between 6.5 to 6.7%.

Majuar (2003) showed that the use of rice husk ash as a pozzolanic material and as a cement replacement improves the splitting tensile strength. In addition, the study indicated that the splitting tensile to compressive strength ratios of high strength concrete range between 5.6 to 7.4%.

Table 4.8: Splitting tensile strength for all mixes

Symbol	Splitting tensile strength (MPa)		
	28d	90d	180d
CC20	4.55	5.16	5.15
CG20	4.38	4.49	4.76
CC14	4.72	5.22	5.32
CS14	4.56	4.86	5.07
CS10	4.92	5.28	5.39
CS10HF	3.5	4.18	4.22
FA10C20	5.02	5.17	5.3
FA10G20	4.79	4.93	5.13
FA10C14	5.14	5.3	5.54
FA10S14	4.78	5.02	5.15
FA10S10	5.2	5.33	5.66
FA10S10FR	4.21	4.51	4.55
LP20C20	5.47	5.94	5.95
LP20G20	5.35	5.62	5.76
LP20C14	5.78	6.11	6.17
LP20S14	5.42	5.91	5.99
LP20S10	5.89	6.25	6.31
MK10C20	5.32	5.58	5.61
MK10G20	5.09	5.21	5.3
MK10C14	5.4	5.63	5.88
MK10S14	5.21	5.33	5.41
MK10S10	5.59	5.91	5.95

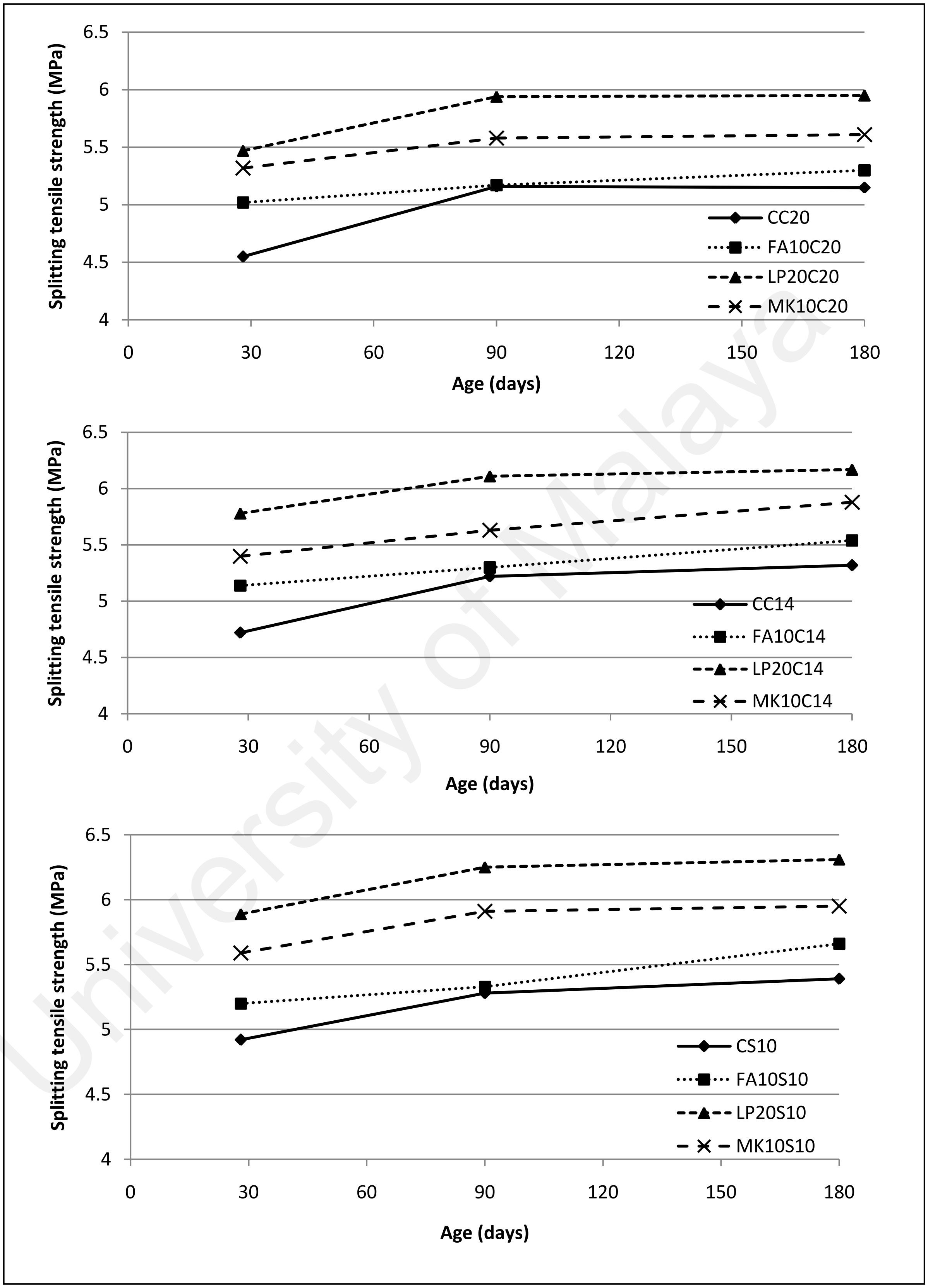


Figure 4.81: Splitting tensile strength of different mineral admixture mixes

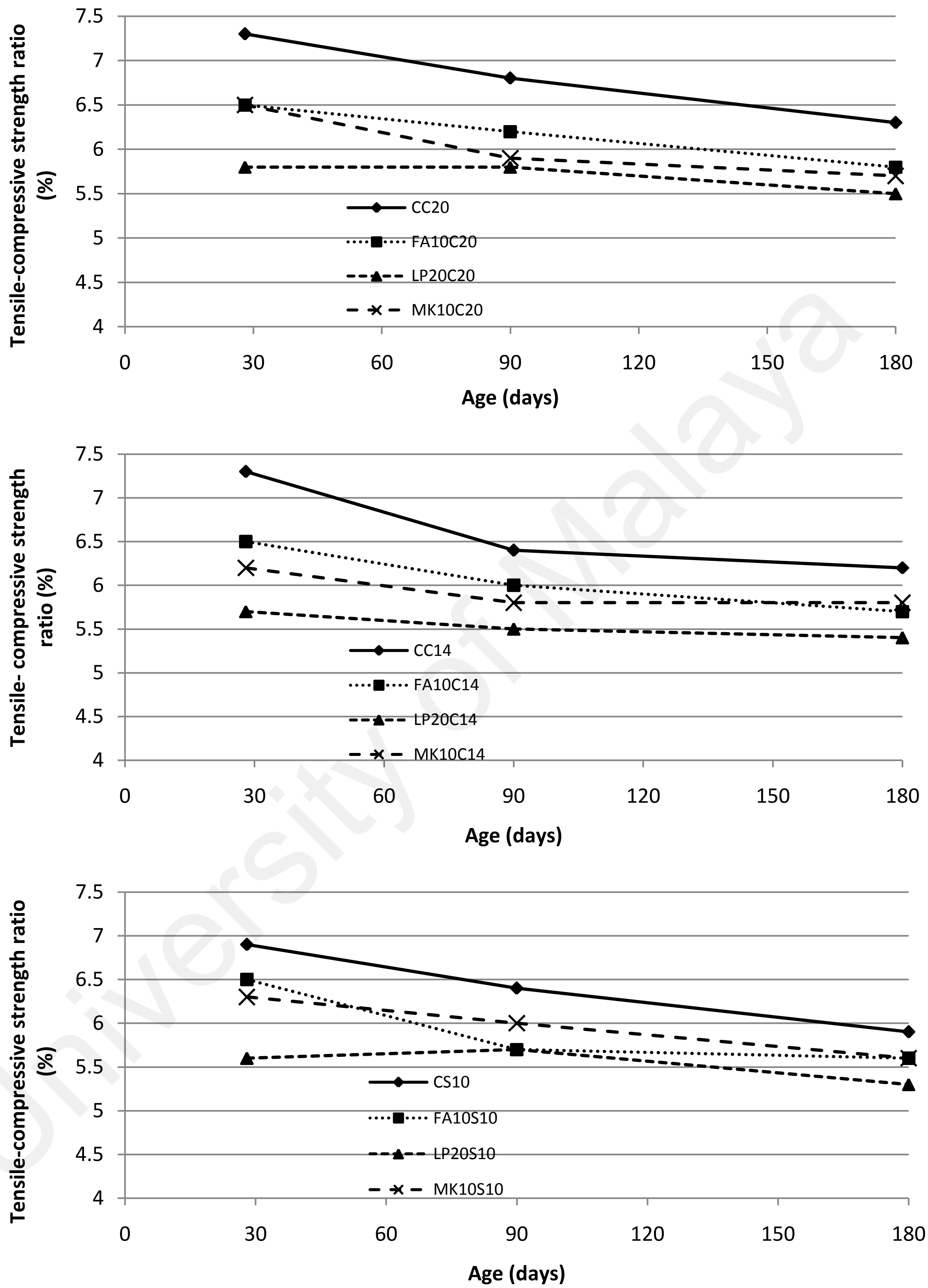


Figure 4.82: Tensile- compressive strength ratio of different mineral admixture concrete mixes

Flexural tensile strength is dependent on compressive strength whereby, the tensile and flexural strength increase with increasing compressive strength. Flexural strength is affected by the characteristics of the interfacial transition zone more than compressive strength. Therefore, the increase in the rate of flexural strength is less than the increase in rate of compressive strength (Mehta & Monteiro, 2006). In cases of flexural strength test, the failure would occur at the interfacial transition zone.

The results of the flexural strength of the mixes are listed in Table 4.9. It is evident from Figure 4.83 that the presence of limestone as replacement of sand in mixes exhibit higher values than the control mixes at all ages. High values of flexural strength of limestone mixes possibly resulted from the filler effect of limestone powder which increases the packing density of paste matrix and leads to a better development of bond due to more advanced cement hydration. This agrees with Nehdi (1998). Conversely, Escadeillas (1988) found that the use of limestone filler as a replacement for cement leads to a reduction in flexural strength at 28 and 90 days. Table 4.10 shows that flexural to compressive strength of limestone mixes ratios vary between 9.6 to 10.6 %.

Figure 4.83 indicates that metakaolin and fly ash mixes gave higher flexural strength when compared with control mixes due to the pozzolanic reactions of metakaolin and fly ash. These reactions enhance the density of the interfacial transition zone by consuming $Ca(OH)_2$ crystals and producing more C-S-H (Habeeb, 2009; Nehdi, 1998; Zhang & Malhotra, 1996). The interfacial transition zone is considered as a major factor in improving the flexural strength. However, flexural to compressive strength ratios of metakaolin mixes range between 10.7 to 11.2% at 28 days while flexural to compressive strength ratios of fly ash mixes range between 11.5 to 12.2% at 28 days. Ratios at the ages of 90 and 180 days are shown in Table 4.10. Generally, it is observed from Table 4.10 that flexural to compressive strength ratios reduce with the increase of compressive strength of concrete and age. This is in agreement with Majuar (2003).

Table 4.9: Flexural tensile strength of all mixes

Mix No.	Flexural strength (MPa)		
	28d	90d	180d
CC20	8.16	8.61	8.89
CG20	7.76	8.34	8.39
CC14	8.28	8.66	9.15
CS14	8.1	8.47	8.6
CS10	8.19	8.81	9.12
CS10HF	6.67	7.32	7.72
FA10C20	8.85	8.88	9.26
FA10G20	8.63	8.79	8.93
FA10C14	9.05	9.19	9.44
FA10S14	8.69	8.73	9
FA10S10	9.22	9.32	9.62
FA10S10FR	7.22	7.77	8.09
LP20C20	9.66	9.79	9.89
LP20G20	9.44	9.48	9.59
LP20C14	9.82	9.93	10
LP20S14	9.66	9.62	9.83
LP20S10	10.13	10.13	10.29
MK10C20	9	9.2	9.52
MK10G20	8.72	8.91	9.12
MK10C14	9.27	9.36	9.62
MK10S14	8.99	9	9.33
MK10S10	9.51	9.49	9.83

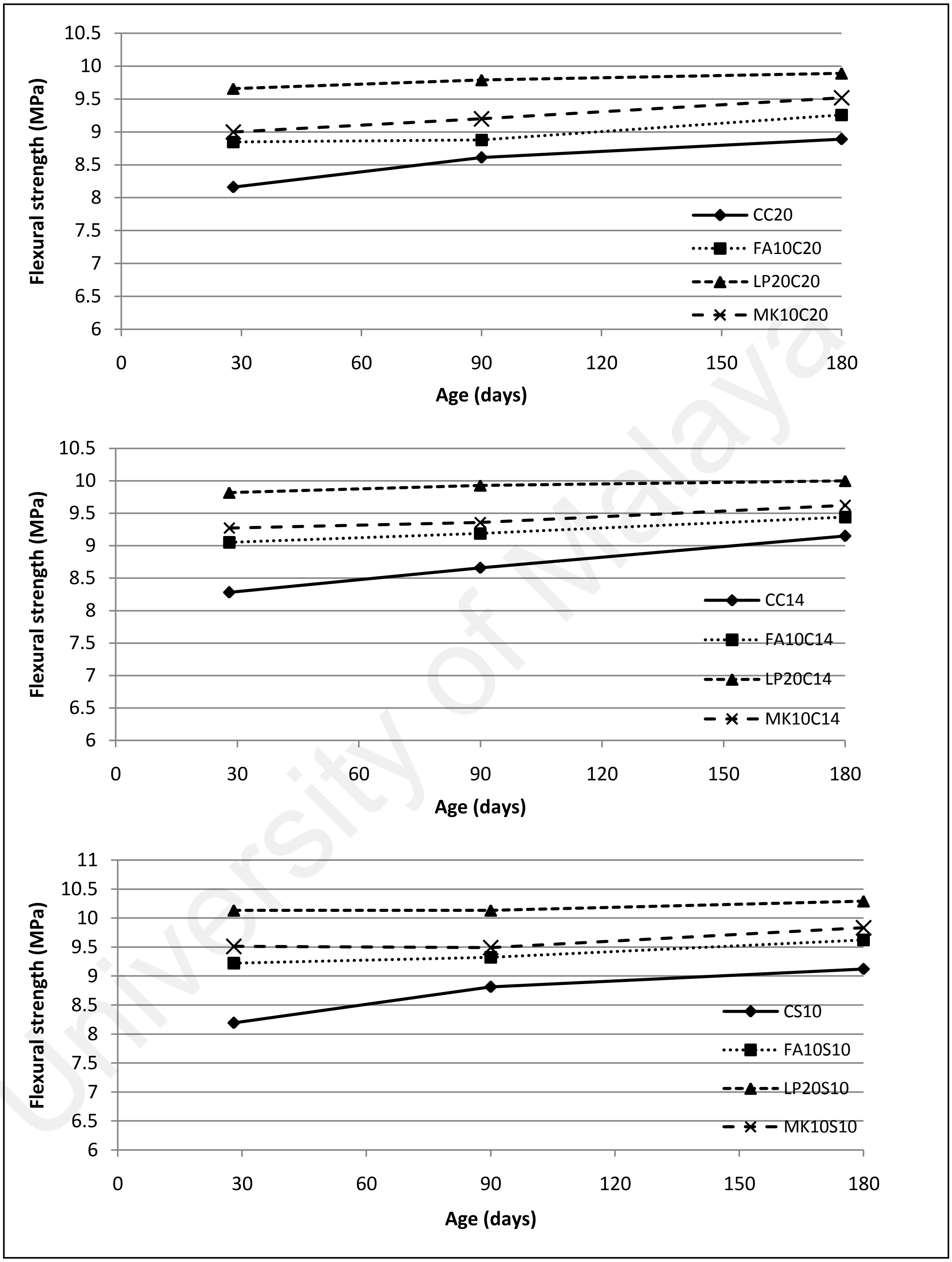


Figure 4.83: Flexural tensile strength of different mineral admixture mixes

Table 4.10: Flexural/compressive strength ratio (%)

Mix No.	α_f/α_c		
	28d	90d	180d
CC20	13.2	11.3	10.8
CG20	13.2	12.4	11
CC14	12.7	10.7	10.5
CS14	13.3	11.8	10.8
CS10	11.5	10.6	10
CS10HFR	14.2	12.4	12.7
FA10C20	11.5	10.6	10.1
FA10G20	11.8	11.1	10.1
FA10C14	11.5	10.4	9.6
FA10S14	12.2	10.8	10.2
FA10S10	11.5	9.9	9.5
FA10S10FR	12.2	10.6	10.5
LP20C20	10.3	9.5	9.2
LP20G20	10.6	9.7	9.4
LP20C14	9.6	9	8.7
LP20S14	10.2	9.3	9.2
LP20S10	9.7	8.8	8.6
MK10C20	11	9.8	9.7
MK10G20	11.2	10.2	10.1
MK10C14	10.7	9.7	9.4
MK10S14	11.4	10.1	9.8
MK10S10	10.7	9.6	9.2

The modulus of elasticity of concrete increases with an increase in the density and volume of coarse aggregate whilst it decreases with an increase in the cement paste content or increasing porosity. There are factors which affect the modulus of elasticity and related to the cement paste structure such as the presence of mineral admixture, air content, degree of hydration and w/b ratio. In addition, the modulus of elasticity is affected by the transition zone between aggregate and paste matrix.

Figure 4.84 indicates the modulus of elasticity of all mixes at 28 and 180 days. Results show modulus of elasticity values of all the mineral powder mixes greater than the modulus of elasticity of the control mix. The modulus of elasticity of filler and pozzolanic mixes vary between 35.5 to 40 GPa according to the type of pozzolan and filler used. The modulus of elasticity of control mixes range between 34 to 35 GPa. The use of limestone powder gives modulus of elasticity of limestone concrete higher than the modulus of elasticity of control mixes by approximately 11 to 13% and 5 to 9% at 28 and 180 days respectively and is shown in Figure 4.85, although the limestone concrete has paste content higher than control concrete. This could be due to the densification which occurs with regards to the density of the limestone concrete paste. Indeed, limestone powder acts as filler to fill voids between cement particles and thus reduce the porosity of the mixes. In addition to this, the effect of limestone on cement hydration acceleration at the early ages leads to an improvement of the paste matrix density and a reduction of the pores in the paste structure. This acceleration effect compensates for the reduction in stiffness which occurs due to greater paste content (Parra et al., 2011). The reduced porosity leads to reduction in the strain and thus an increase in the modulus of elasticity.

Metakaolin mixes exhibit higher modulus of elasticity than control mixes by approximately 6 to 9% and 3 to 5% at 28 and 180 days respectively as shown in Figure 4.85. The pozzolanic activity of metakaolin plays an important role in improving the transition zone between paste and aggregate by producing more C-S-H and thus making this zone denser than that in the control mixes.

The fly ash mixes have modulus of elasticity higher than that of the control mixes. Indeed, the percentage enhancement varies between 5 to 7% at 28 days and between 1 to 3% at 180 days when compared to the control mix.

The percentage enhancement of fly ash with control mixes is lower than the percentage increase of metakaolin with control mix. This could be due to the fact that the paste volume of fly ash mixes is more than the paste volume of metakaolin mixes. It could also be due to the superior pozzolanic activity of metakaolin when compared to the fly ash mixes.

Sata et al. (2007) concluded that the use of fly ash in high strength concrete has a slight effect on modulus of elasticity. Moreover, the modulus of elasticity of pozzolanic mixes increases with an increase in compressive strength. Roziere et al. (2007) mentioned that in self compacting concrete the modulus of elasticity reduces with an increase in paste volume. This can be explained by the paste being less stiff than aggregate and as paste volume increased meant that aggregate volume decreased.

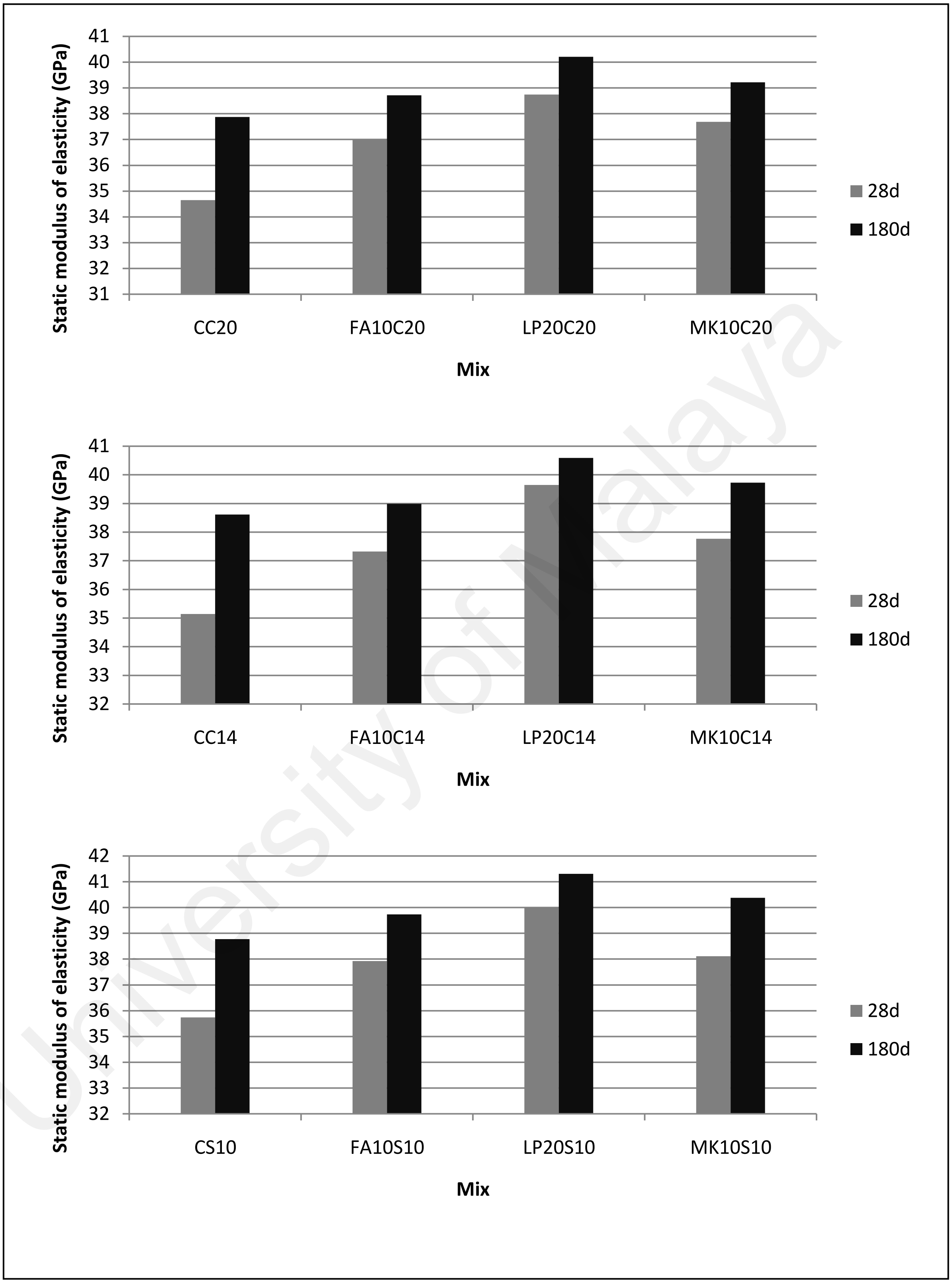


Figure 4.84: Modulus of elasticity of different mineral admixture mixes

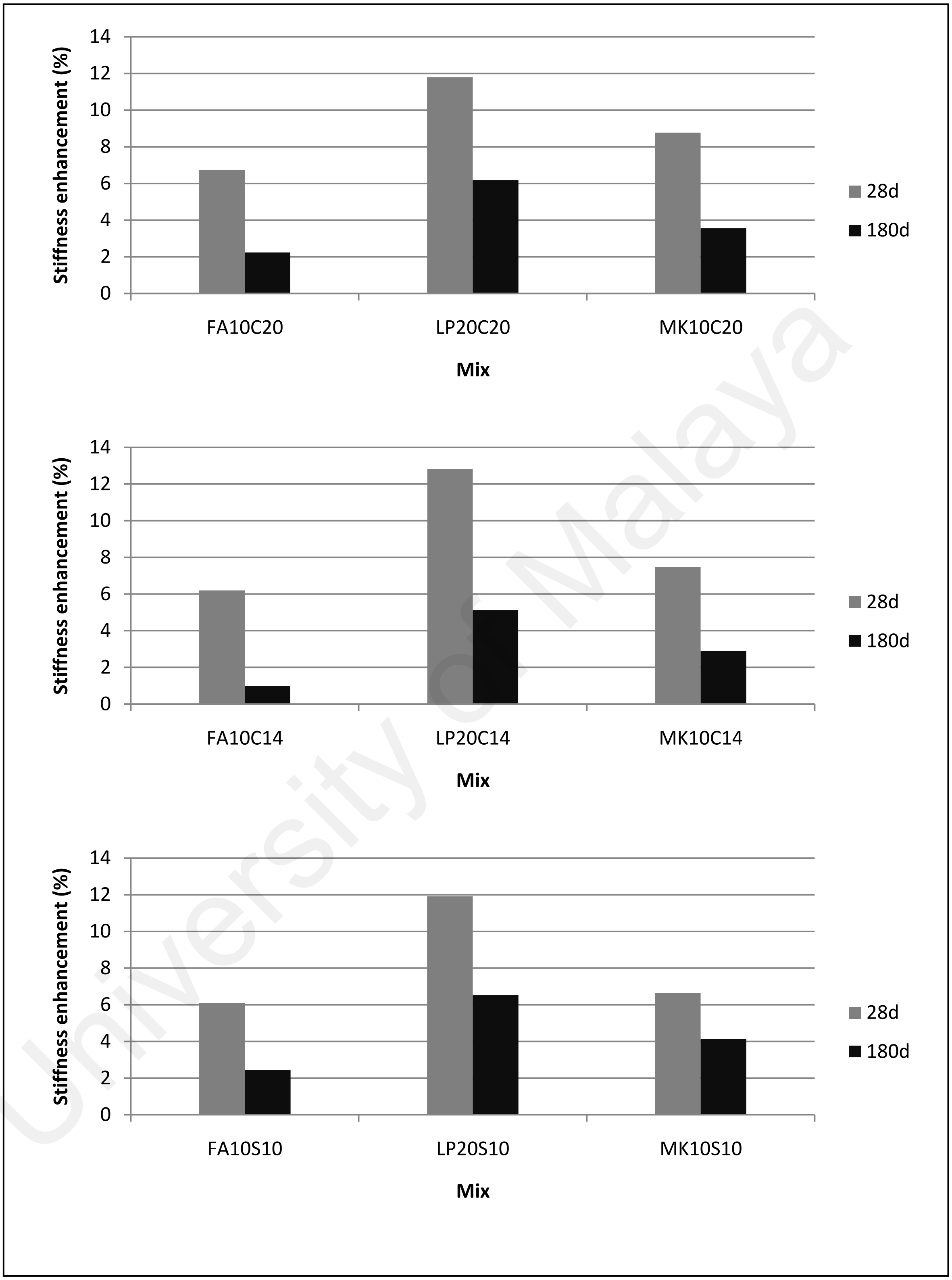


Figure 4.85: Stiffness enhancement of different mineral mixes with control mix

The water absorption results of all mixes at the ages of 28, 90 and 180 days are listed in Table 4.11. The range of the results varies from 2.22 to 6.32% for all mixes at 180 days which are low. These low water absorption results indicate that the quality of concrete is good. Generally, it can be seen from Figure 4.86 that there is a significant reduction in the values of the water absorption of mixes which contain pozzolan or filler when compared with control mixes. The limestone mixes show lower water absorption values when compared with other mixes. Selvamony et al. (2010) reported that the use of limestone powder improves durability of concrete by producing a more compact paste structure and filling the pores between cement particles. Beixing et al. (2009) stated that the employment of limestone fines as partial replacement for sand has a slight influence in improving the durability of high strength manufactured sand concrete. The results show that the water absorption of metakaolin mixes is less thus giving a pozzolanic effect superior to fly ash. This pozzolanic effect makes a denser concrete and improves the particle packing density which reduces the volume of large pores and segments the pores thus hindering the movement of water through the paste structure (Safiuddin, 2008; Bentz & Haecker, 1999; Parrott, 1992) when compared to the water absorption of fly ash mixes. This could be due to the fact that the pozzolanic activity of metakaolin in producing C-S-H is more than that of fly ash and thus reduces the capillary pores. Furthermore, the fineness of metakaolin is more than the fineness of fly ash.

Table 4.11: Water absorption of all mixes

Symbol	Water absorption (%)		
	28d	90d	180d
CC20	4.33	4.07	3.75
CG20	4.5	4.21	3.8
CC14	4.26	3.95	3.6
CS14	4.43	4.19	3.85
CS10	4.31	3.85	3.5
CS10HFR	6.77	6.5	6.32
FA10C20	3.96	3.5	2.95
FA10G20	4.15	3.76	3.14
FA10C14	3.83	3.43	3
FA10S14	4.1	3.78	3.12
FA10S10	3.9	3.37	2.69
FA10S10FR	5.28	4.71	4.44
LP20C20	2.99	2.65	2.47
LP20G20	3.12	2.74	2.61
LP20C14	2.87	2.52	2.34
LP20S14	3.07	2.93	2.72
LP20S10	2.72	2.5	2.36
MK10C20	3.19	2.94	2.7
MK10G20	3.33	3.12	3
MK10C14	3.11	2.93	2.79
MK10S14	3.41	3.12	2.91
MK10S10	3	2.79	2.22

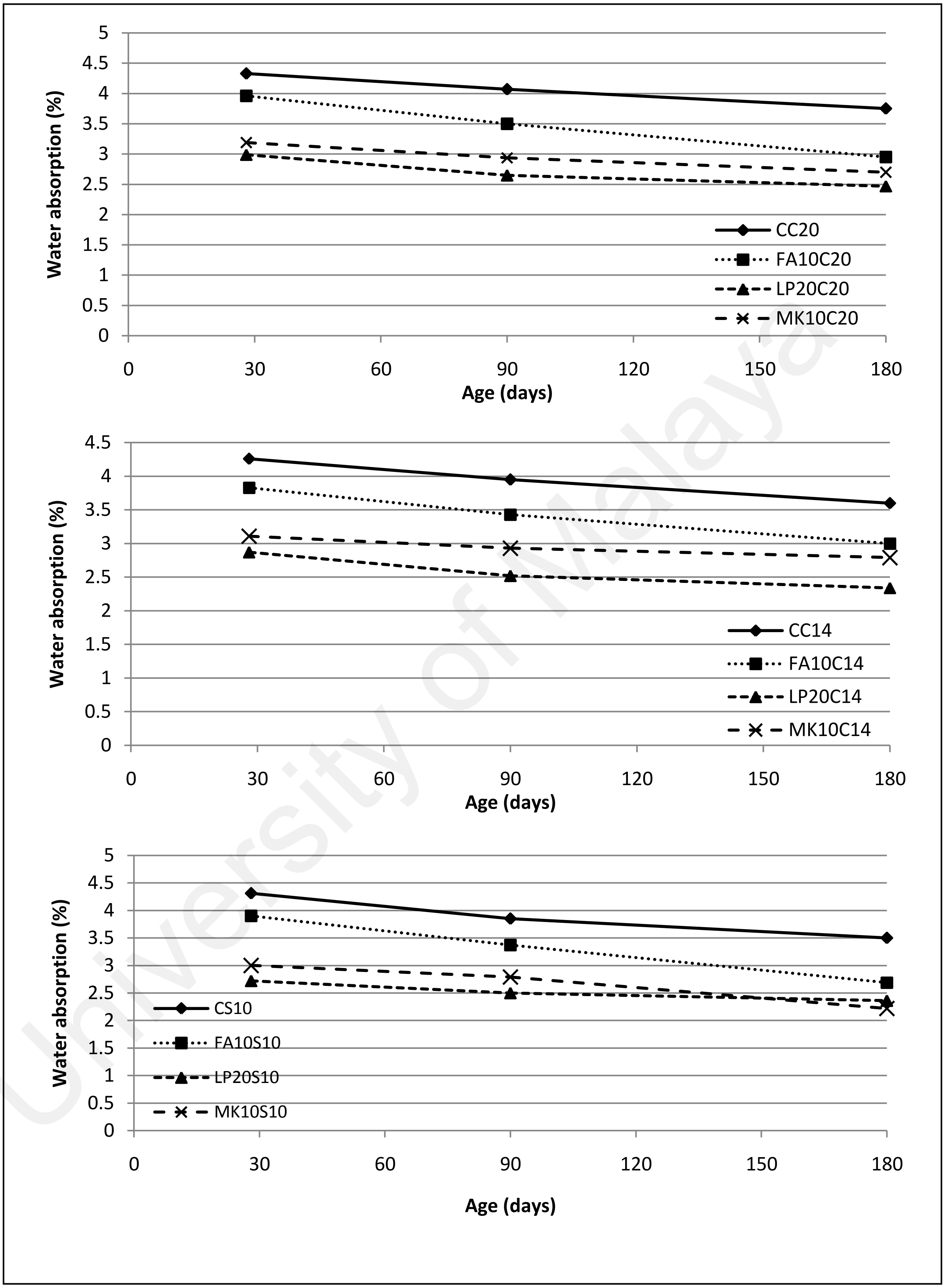


Figure 4.86: Water absorption of different mineral admixture mixes

There are many factors, both internal and external, which affect drying shrinkage. The external factors for all mixes are identical in this study. The internal factors are related to the constituents of concrete such as cement content, cement composite, pozzolanic and filler content, aggregate volume, water content, etc. In this section, the effect of mineral additives on drying shrinkage will be discussed, related to either cement or sand replacement. Generally, the results of all mixes in the present study are high and this is due to the high cement content and relatively low coarse aggregate content when compared with the shrinkage values of high or normal strength concrete. The effect of pozzolans and fillers on drying shrinkage of concrete is different and varies according to their chemical and physical characteristics. The chemical effect is due to their pozzolanic reactivity and its relation with the structure of cement paste. The physical effect is due to the behaviour of these additives as they are dispersed. The results of drying shrinkage at different ages of fly ash mixes are shown in Figures 4.87 and 4.88.The figures illustrate that the fly ash mixes have low drying shrinkage when compared to control mixes at early ages until 7 day. After 7 days, the drying shrinkage of fly ash mixes exhibited higher drying shrinkage than that of the early ages. This agrees with Gupta et al. (2006), Mokarem (2002), and Khatri and Sirivivatnanon (1995). As indicated in Figure 4.87 and 4.88, the values of drying shrinkage of MK mixes during the early ages are rather low when compared to control mixes. Indeed, after the first week the results of MK mixes are comparable with the control mixes. This is in agreement with Kinuthia et al. (2000). This may be due to a number of reasons. The pozzolanic activity is expected to be the main factor which affects the drying shrinkage of concrete. The pozzolanic activity leads to a refined pore structure of cement paste, which in turn increase the gel particles and reduces capillary pores in the cement paste. Gel particles contain adsorbed water on their surface thus the loss of this adsorbed water will lead to higher shrinkage of concrete. Moreover, the specific gravity of pozzolanic materials is lower than that of cement leading to an increase in the volume of paste which in turn increases the shrinkage.

In the literature, the results of the effect of mineral additives on drying shrinkage are rather contradictory. Several researchers have found that the use of pozzolanic additives leads to an increase in drying shrinkage of concrete. In contrast, others have found that these additives decrease the drying shrinkage of concrete. However, no conclusion can be drawn with regards to these differences of opinions. This difference is due to the various characteristics of the additives. Mehta and Montoero (2006) reported that pozzolans lead to an increase in the volume of the gel pores in the cement paste structure whilst drying shrinkage of concrete is related to the evaporation of water in these pores. Indeed, the high drying shrinkage is due to the pore refinement of cement paste structure. Mokarem et al. (2005) showed that using fly ash in concrete as a cementitious material gives greater drying shrinkage than that of normal cement concrete. This may be attributed to the denser matrix of fly ash concretes which makes the capillary pores smaller. Hence, drying shrinkage would increase due to the removal of the water inside the smaller capillary pores. In a study by Malathy and Subramanian (2007), it was concluded that the drying shrinkage of metakaolin and fly ash mortars are higher than that of Portland cement mortar. The increase is due to the pozzolanic activity which changes the mechanism of pore size refinement. On the other hand, some researchers reported that the replacement of cement with pozzolanic additives will reduce the drying shrinkage (Chindaprasirt, et al., 2004; Guneyisi, 2008; Gesogˇlu M et al., 2009). In the study by Teorenau & Nicolescu (1982), the use of fly ash as a replacement for cement increases the shrinkage more than using Portland cement. This effect increases with higher amounts of fly ash. Brooks &

Johari (2001) concluded that metakaolin concrete yields lower values of shrinkage when compared with plain concrete.

Figures 4.87 and 4.88 reveal that the limestone mixes have higher shrinkage than that of the control mixes, which may be attributed to the fact that the limestone mixes have high powder content compared to control mixes. Moreover, limestone in the mixes is used as a partial replacement for sand, which means that the amount of sand in limestone mixes is less than that in control mixes. Indeed, sand particles restrain the shrinkage better than limestone powder due to the fact that sand particles are coarser than LP particles. In addition to this, high fineness of limestone has led to increase C-S-H formation due to nucleation effects of limestone particles. This in turn increases the gel particles in the cement paste thus leading to an increase in the shrinkage, the results are in agreement with Ahmed & El-Kourd (1989), Cochet & Sorrentino (1993), Nehdi (1998).

Adams & Race (1990) found that using limestone powder as both fine and coarse fillers at different levels of replacement leads to an increase in the shrinkage on the 4th day. This can be attributed to the increase in fineness of limestone as well as increasing the C-S-H creation at early ages due to nucleation effects. Hence, the gel particles in the cement paste are increased as well as the amount of shrinkage (Nehdi, 1998).

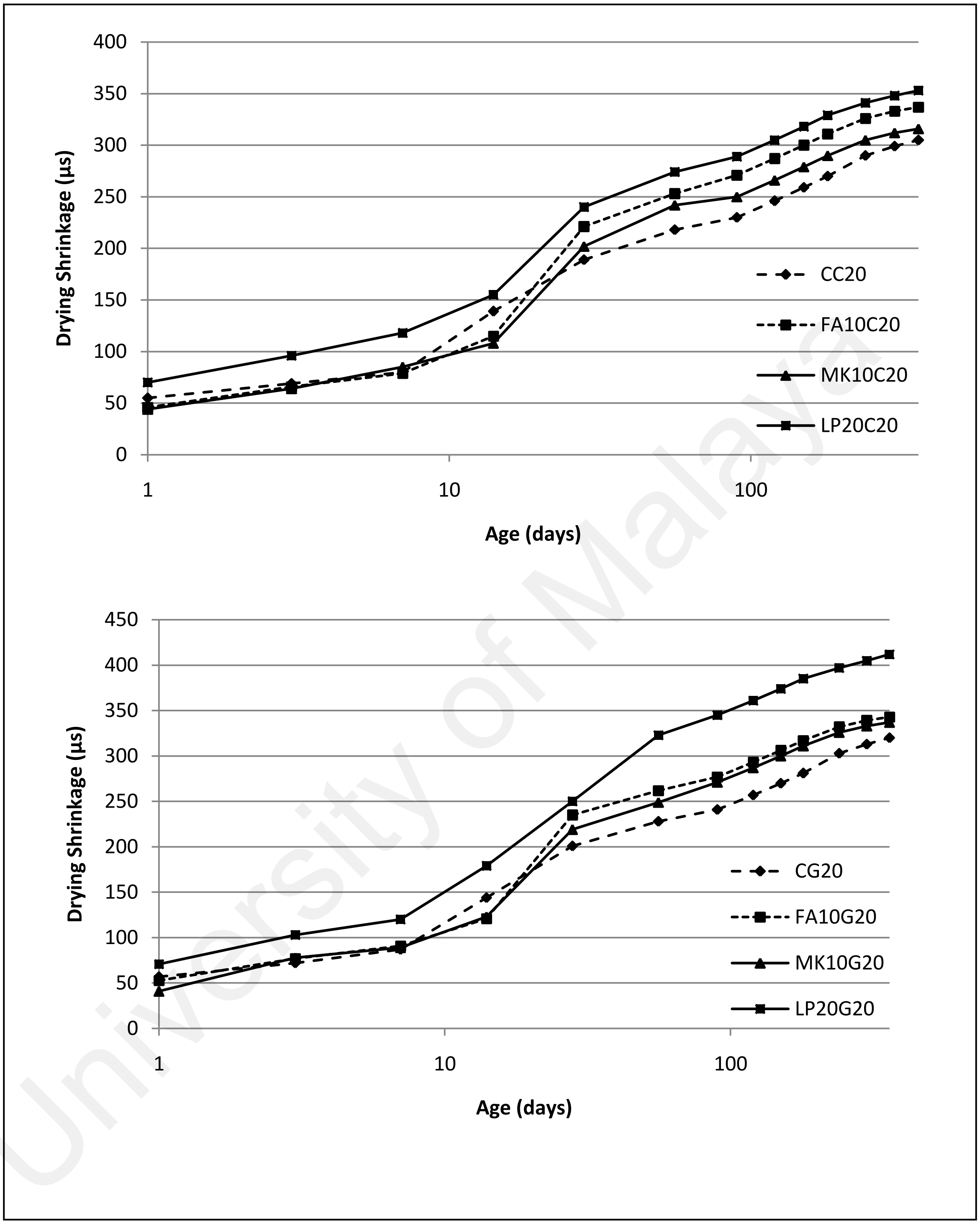


Figure 4.87: Drying shrinkage of different mineral admixture mixes

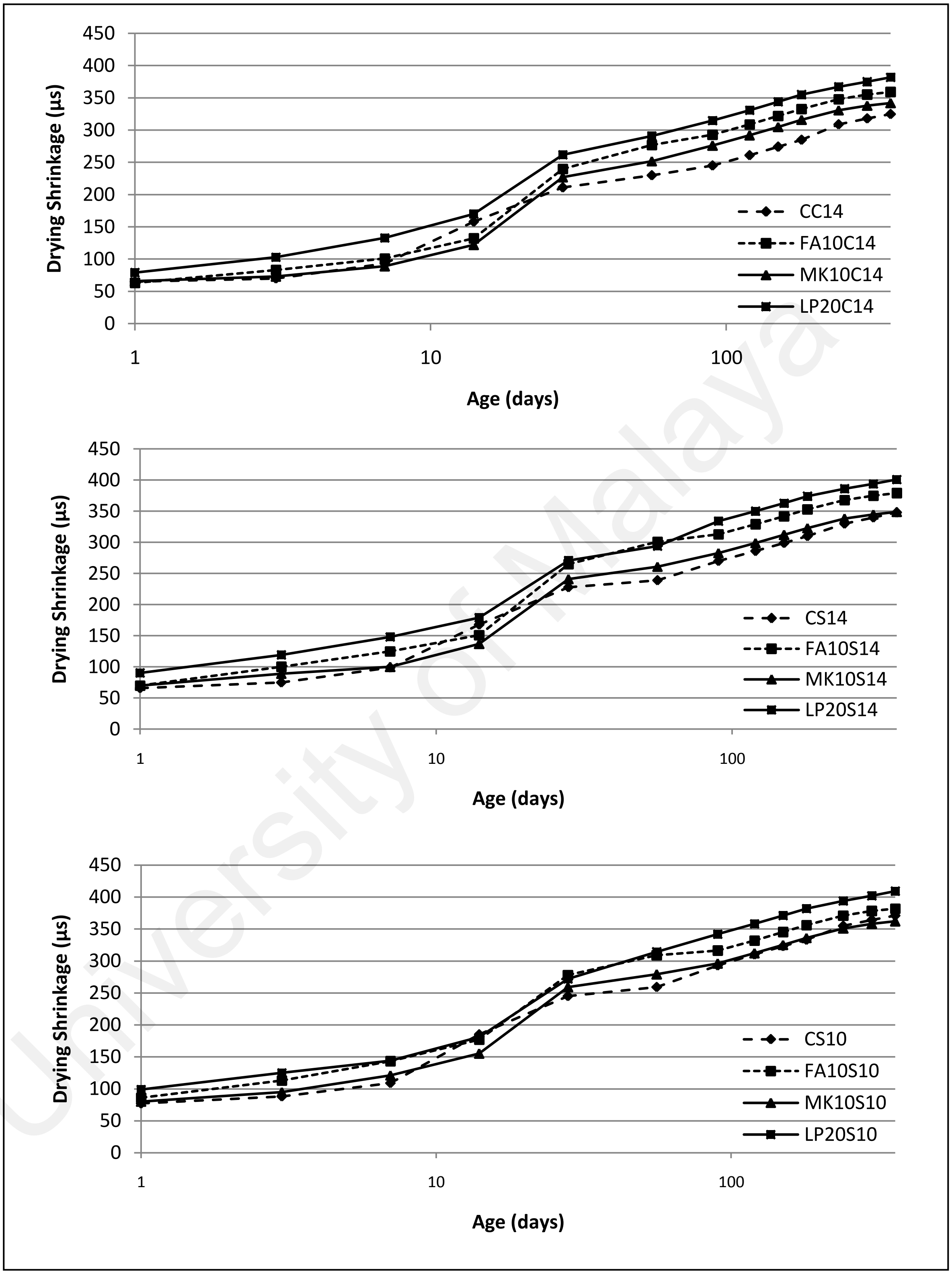


Figure 4.88: Drying shrinkage of different mineral admixture mixes

The results indicate that ultra pulse velocity (UPV) values increase with the increase of age as shown in Figure 4.89. It can be observed clearly that the presence of limestone powder in the mixes increase UPV by approximately 16.3 to 18.3% when compared with the control mixes. This is mainly due to the fact that the use of limestone powder enhances the quality of the mixes by decreasing the pore size, number and densification of pore structure of concrete mixes (Safuiddin, 2008). The filling effect of limestone powder plays a major role in increasing the density of paste matrix and interfacial transition zone which in turn increases the UPV values. In addition, Figure 4.89 demonstrates that the mixes containing metakaolin and fly ash exhibit UPV values higher than the control mixes. The percentage increases vary between 9.5 to 12.5% of metakaolin mixes and 5 to 9% of fly ash mixes. This increase results from the aforementioned pozzolanic activity which results in a reaction between pozzolanic materials and cement hydration products such as Ca $(OH)_2$ crystals. This leads to the production of CSH and the filling of the pores in the cement matrix and transition zone between aggregate and cement.

It was observed that the percentage increase of metakaolin mixes is more than that of fly ash mixes. This could be due to the fact that the fly ash has a low specific gravity which increases the paste volume and decreases the aggregate volume resulting in a reduction in UPV values of fly ash mixes when compared with metakaolin mixes (Safiuddin, 2008; Naik & Malhotra, 1991; Shetty, 2001).

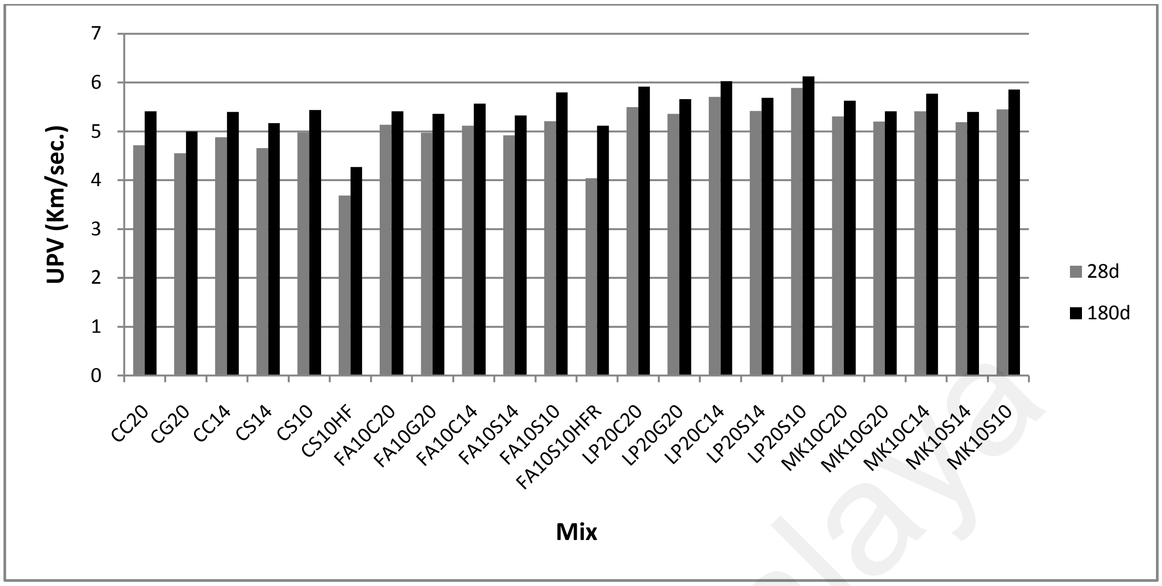


Figure 4.89: UPV values of all mixes

4.7.2 Effect of coarse aggregate volume

The effect of an aggregate volume on the compressive strength of concrete is shown in Figure 4.90. From this figure, strength of concrete increases slightly with increasing amount of coarse aggregate. This increase in strength of concrete stopped when the amount of coarse aggregate in the concrete mixture became 33%. However, the strength of concrete reduces when the amount of coarse aggregate exceeds 33% in concrete. The percentage decrease of strength of mixes with coarse aggregate content of 29% and 31% are 8.77% and 5.1% respectively compared with the strength of concrete with 33% coarse aggregate content. This may be attributed to the fact that microcracks propagate in high strength concrete as described in detail and as follows i.e. where the nature of the crack through the matrix is finer than the crack in concrete due to an increase of the amount of obstacles where these obstacles deviate from the path of these cracks and become longer. In addition, the bond strength between aggregate and cement paste in high strength concrete is strong, and it becomes difficult for cracks to penetrate it. Therefore, as the coarse aggregate increases, this leads to an increase in the path of cracks which in turn increases the fracture energy required for failure. However, at higher content of coarse aggregate, the matrix content is lower and the strength of transition zone is less. Consequently, when there is much large content of coarse aggregate, it leads to a decrease in the compressive strength. This was observed by Cetin and Carrasquillo (1998), Feng et al. (1995), De Larrard et al. (1997) and Chen and Liu (2004). Suwanvitaya et al. (2006) concluded that adding more coarse aggregate increases the strength slightly and this increment reduces as w/c ratio increases. Amparano et al. (2000) studied the effect of coarse aggregate content on the properties of concrete. The study mentioned that in high strength concrete case, the cement paste could be even stronger than aggregate, thus the increase in coarse aggregate volume would impair the concrete strength. Cetin and Carrasquillo (1998) investigated the effect of different amount of coarse aggregate on compressive and tensile strength of high strength concrete with a w/c ratio equivalent to 0.28. The study consisted of using three different amounts of coarse aggregate of 36%, 40% and 44% of concrete volume. The authors have observed that the optimum volume ratio was between 36 to 40% and beyond 40% the strengths would reduce. Meddah et al. (2010) investigated the effect of different coarse aggregate contents namely 44% to 47% on the compressive strength of high and normal strength concrete. In the case of high strength concrete, the results indicated that the compressive strength of concrete increased with increase of coarse aggregate content from 44% to 45%. While, beyond 45% the compressive strength decreased with increase of coarse aggregate content. In the case of normal strength concrete, the results showed the compressive strength increase as coarse aggregate content increase so that 47% was the optimum volume content.

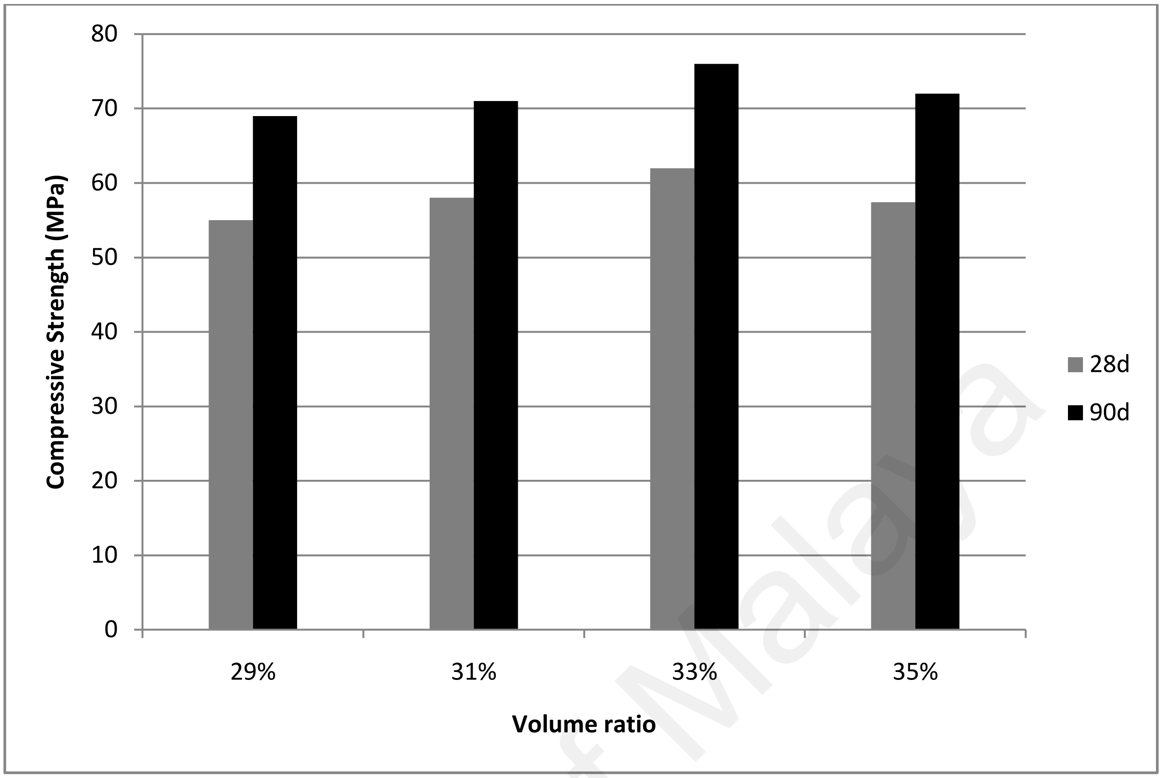


Figure 4.90: Compressive strength with different aggregate contents

The effect of the volume ratio of coarse aggregate on splitting tensile strength is shown in Figure 4.91. The results at 28 and 90 days reveal that the strength increases as the volume ratio of coarse aggregate in the mixes increases. The possible explanation for this increase is attributed to the interface transition zone around the coarse aggregate, which is deemed to be the strong zone in high strength concrete. Adding more aggregate beyond 33% leads to a decrease in strength. The reason could be due to the increase of coarse aggregate volume relative to total volume of concrete lead to an increase in the difference between modulus of elasticity of aggregate and matrix. This causes weakening of the transition zone between aggregate and mortar which in turn leads to a reduction in tensile strength of mixes. Moreover, the increased coarse aggregate volume reduces compactability of mixes and in turn affects the strength of concrete. This was also deduced by Akçaoğlu et al. (2004).

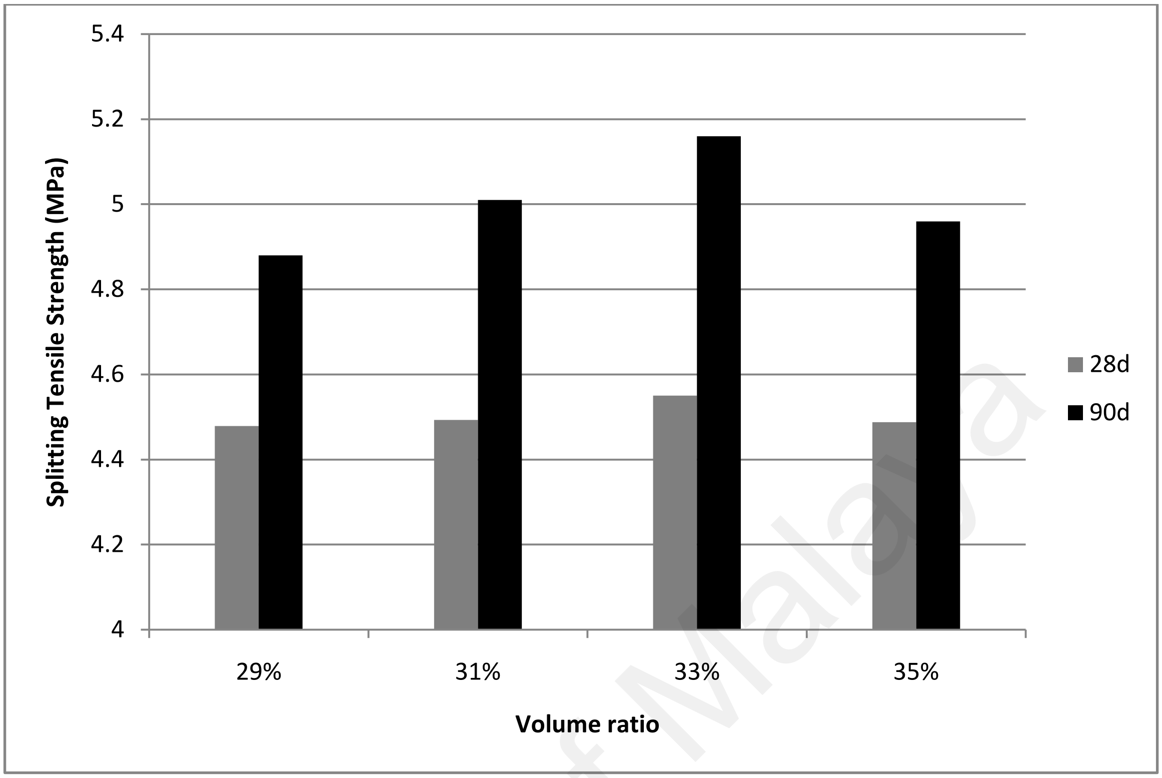


Figure 4.91: Splitting tensile strength of different aggregate contents

Figure 4.92 shows the effect of coarse aggregate volume on the flexural strength of concrete mixes at 28 and 90 days. The results show the flexural strength increase with an increase of coarse aggregate volume in the mixes until 33%. The flexural strength of mixes with coarse aggregate content more than 33% was reduced. The behaviour of results pertaining to flexural strength of the mixes is compatible with the behaviour of results regarding compressive strength and tensile strength. This could due to the fact that in high-strength concrete, the transition zone between the matrix and coarse aggregate is sufficiently strong, and the tension in concrete is significantly affected by this zone. However, the cracks' propagation in concrete under load in flexural passes through coarse aggregate particles. With this, the increase of coarse aggregate content leads to an increase in the transition zone meaning that the strength will increase. However, the reason of reduction of flexure strength of mixes with aggregate more than 33% could be attributed to the fact that the distance between adjacent aggregate particles is very close and thus the cracks propagate faster.

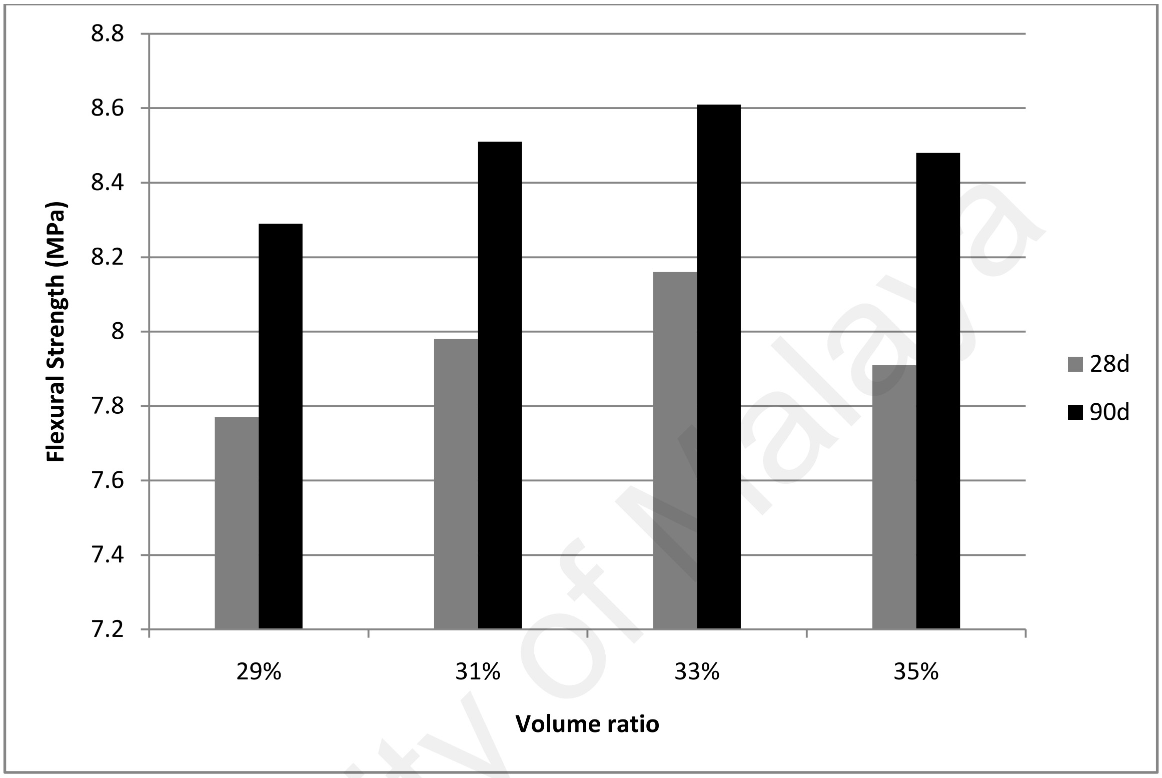


Figure 4.92: Flexural strength of different aggregate contents

Figure 4.93 illustrates that the modulus of elasticity of mixes at 28 and 90 days increases with an increase coarse aggregate volume until it has a coarse aggregate volume of 33% before the modulus of elasticity drops. This increase could result from increasing aggregate fractions so that the difference between modulus of elasticity of both the matrix and aggregate will be reduced. This leads to improved bond strength of the interfacial zone and thus modulus of elasticity is better. In contrast, reduction occurs with aggregate fractions more than 33% of concrete volume. This could be attributed to the fact that the increase of aggregate volume leads to an increase in the difference between modulus of elasticity of both matrix and aggregate. This in turn reduces the modulus of elasticity of concrete.

Aulia and Deutschmann (1999) reported that the difference between modulus of elasticity of cement paste and aggregate affect the bond stress at transition zone. As the difference in the modulus of elasticity of the two matrix increases, the stress concentration at the zone is higher and thus reduces the modulus of elasticity of concrete.

Karam (1997) found that as the coarse aggregate volume increase there is a simultaneous increase in ductility of high strength concrete due to an increase in zigzagging with high coarse aggregate volume. Meddah et al. (2010) investigated the effect of different coarse aggregate contents namely 44% to 47% on the compressive strength and modulus of elasticity of high and normal strength concrete. The results showed modulus of elasticity is proportional to compressive strength as mentioned previously about compressive strength results. Bonen and Shah (2004) reported that concrete with low coarse aggregate contents have low modulus of elasticity. Therefore, this work reported that self compacting concrete has lower modulus of elasticity compared to normal slump concrete of the same strength. Schindler (2007) concluded that the values of modulus of elasticity of SCC mixes at 18 hours were less than that for control mixes with the same compressive strength. At 56 days and beyond, the modulus of elasticity of SCC is approximately similar to that of control mixes with the same compressive strength.

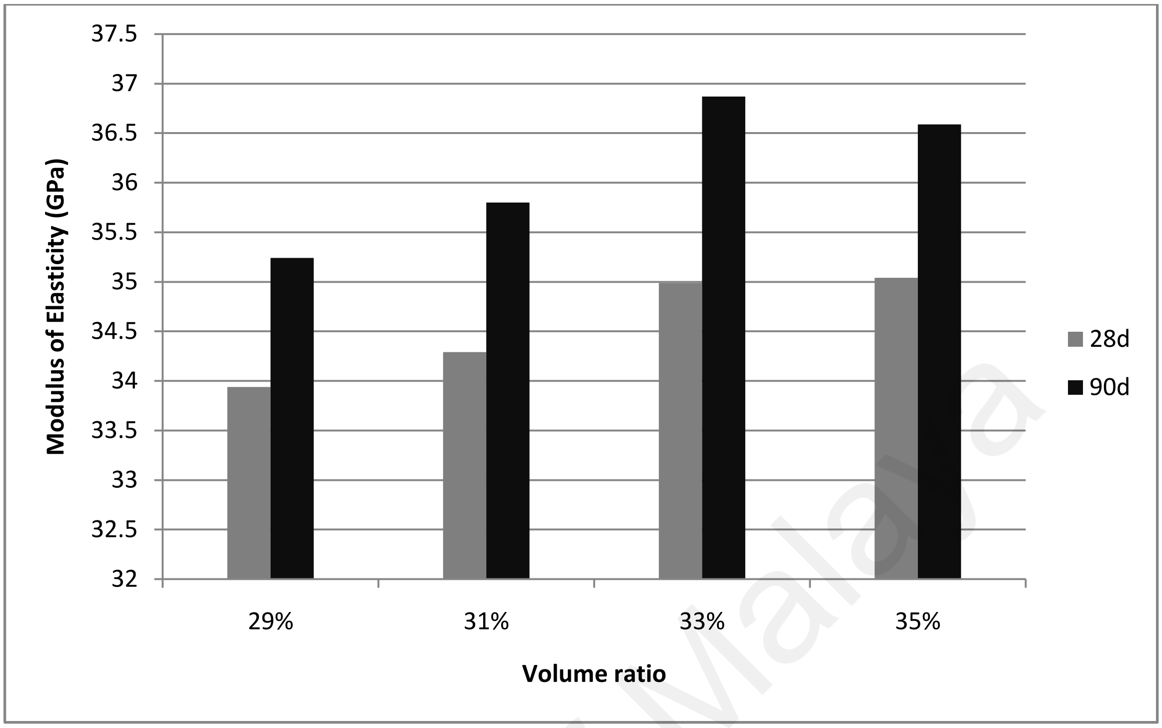


Figure 4.93: Modulus of elasticity of different aggregate contents

The results shown in Figure 4.94 reveal the effect of coarse aggregate volume on water absorption of concrete mixes at 28 days. The results show that the increase in coarse aggregate volume affects water absorption so that the water absorption is reduced. This reduction results from the reduction in paste volume per concrete volume which in turn reduces the porosity of concrete. Therefore, water absorption is reduced which agrees with the study of Buenfeld & Okundi (1998).

Kolias and Georgiou (2005) conclude that capillary absorption is affected by the paste volume as well as w/c ratio. Increase in the paste volume, leads to an increase in water absorption of concrete mixes.

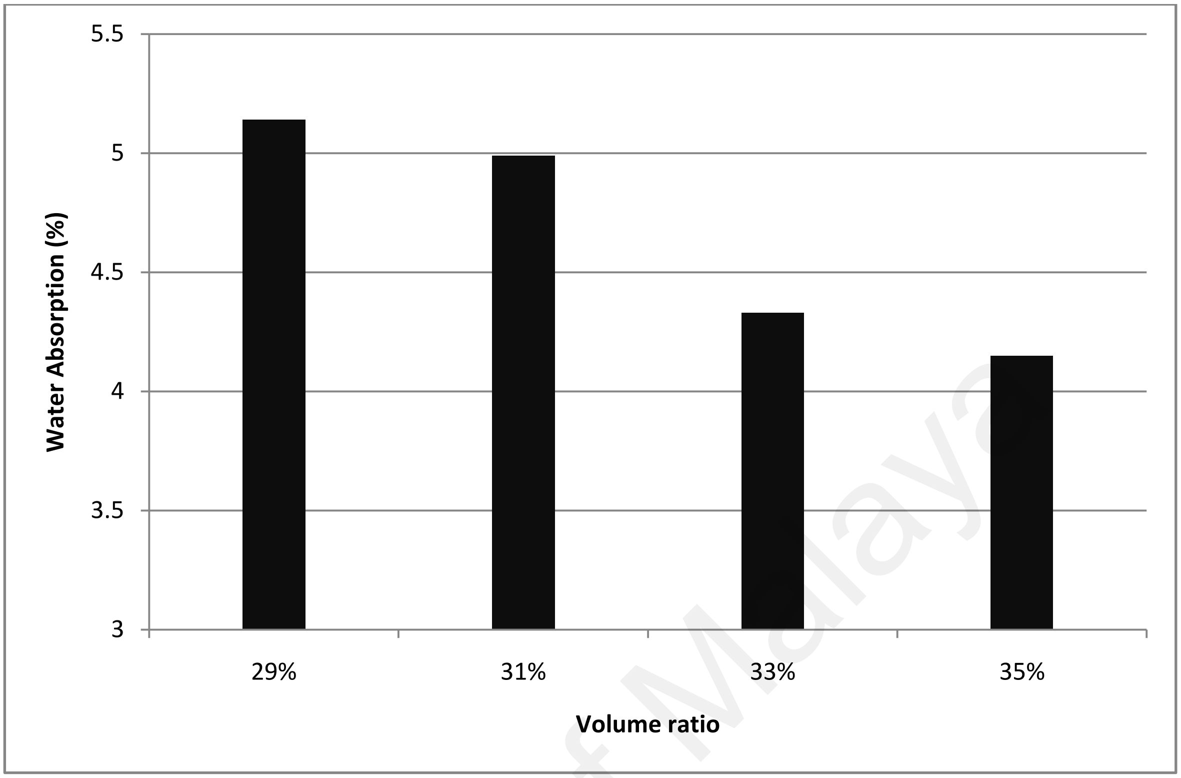


Figure 4.94: Water absorption of different aggregate contents at 28 days

The volume ratio of coarse aggregate in concrete is considered as an internal factor and plays a direct role in affecting shrinkage. Indeed, coarse aggregate with low shrinkage property restrains the shrinkage of cement paste. The increase of coarse aggregate volume means an increase in coarse aggregate/cement paste ratio in concrete. From the results indicated in Figure 4.95, the role of coarse aggregate volume in concrete clearly appears to affect the shrinkage. As the coarse aggregate volume increases, the shrinkage occurring in cement paste is constrained, leading to a decrease in shrinkage of concrete (Aitcin, 1998). This is due to the fact that drying shrinkage is mainly associated with the process of adsorbed water loss which exists in the paste, and a high volume of paste will increase the shrinkage (Kosmatka et al., 2002). Aitcin (1998) emphasises that although an excess of coarse aggregate could decrease drying shrinkage it will increase the amount of microcracks within the paste.

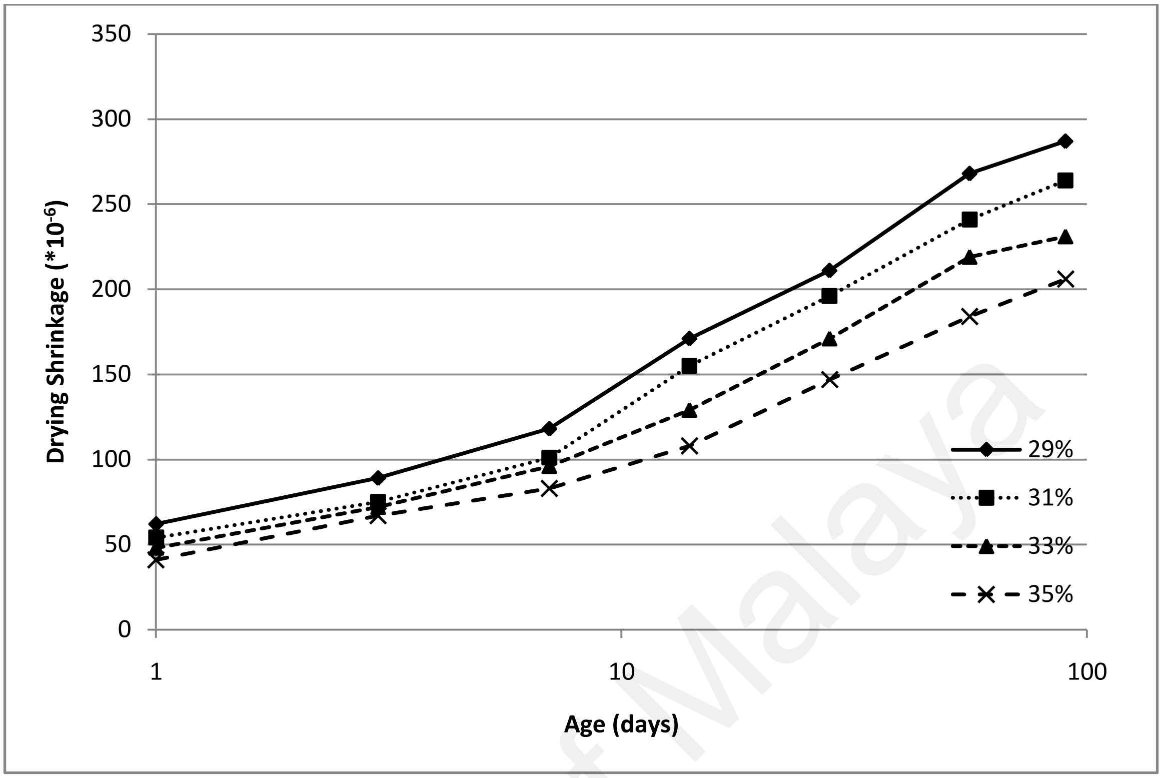


Figure 4.95: Drying shrinkage of mixes with different aggregate ratios

Figure 4.96 indicates the effect of coarse aggregate volume on the values of UPV test. It is apparent that pulse velocity increases with an increase in coarse aggregate volume in concrete mixes. This increase is due to the fact that increasing the coarse aggregate volume in the mixes accelerates the pulse propagation through the sample and thus the values of ultrasonic velocity are increased (Naik & Malhotra, 1991, Shetty, 2001 and Abo-Qudais, 2005). Most of UPV results also indicated a consistent increase with compressive strength values. Whereas, Lin (2003) reported that as cement paste volume increases, the pulse velocity of concrete decreases especially with high levels of water cement ratio.

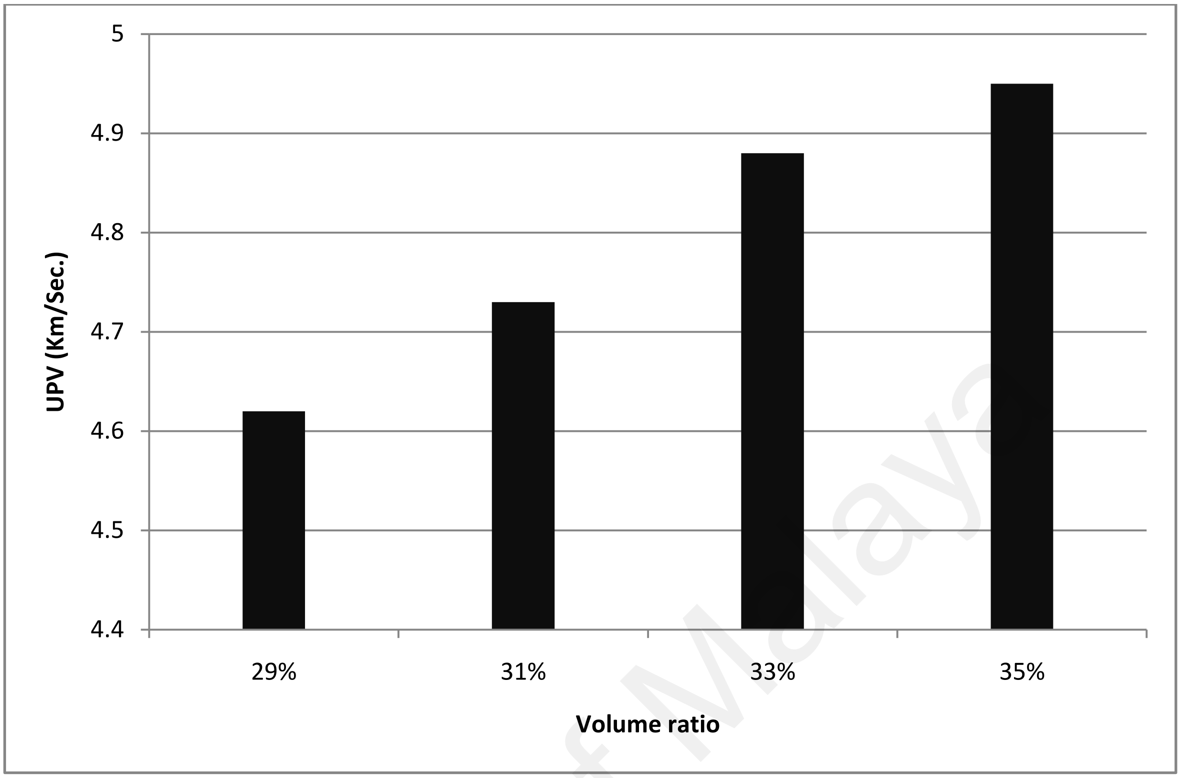


Figure 4.96: UPV of different aggregate ratios

4.7.3 Effect of coarse aggregate maximum size

From the results presented in Figure 4.97, it is noted that the compressive strength of mixes made with 14 mm graded coarse aggregate is higher than the compressive strength of mixes made with 20mm graded coarse aggregate. Relative strength of the effect of maximum size is listed in Table 4.12. At 28 and 180 days of curing, the relative strength of C14 mixes ranges between 1.02 to 1.08 and 1.04 to 1.07 respectively compared with C20 mixes. Moreover, the compressive strength of 10mm single sized aggregate concrete is higher than the compressive strength of 14mm single sized aggregate concrete as shown in Figure 4.97. The relative strength of S10 mixes are between 1.11 to 1.16 and 1.11 to 1.15 when compared to S14 mixes at 28 and 180 days of curing respectively as shown in Table 4.12. This may be attributed to the fact that the transition zone in high strength concrete is strong enough so that the energy required for failing of the sample is high. As such, the small maximum size of coarse aggregate has a larger surface area and thus the transition zone is larger and strength is higher. This is compatible with the results of Aulia & Deutschmann (1999), Meddah et al. (2010).

Washa et al. (1998a) studied the effect of maximum size of aggregate on strength of high performance concrete. This study showed that with high cementitious content and low water binder ratio, the use of large maximum size reduces strength. This could be due to that bond at transition zone with large particles is weaker than that with small particles because the surface area to volume ratios in large particle case is smaller.

Yan et al. (2001) investigated the effect of maximum size aggregate on the fracture details of high strength concrete. This paper found out that at the same mortar strength, the dimensions of fracture increase as the maximum size of aggregate increased. This study also reported that the fracture dimension increase as the compressive strength of concrete decreased.

Tasdemir et al. (1996) investigated the effect of silica fume replacement with cement and maximum size aggregate on fracture energy and brittleness index of high strength concrete. This work found out that the fracture energy of concrete reduced when the large aggregate size was used and thus the brittleness index of concrete increased.

Khaleel et al. (2011) studied the effect of type and maximum size of coarse aggregate on self compacting concrete properties at the same mix proportions and w/c ratio of 0.34. The results showed that the decrease in maximum size led to increase in compressive strength of mixes of different types of aggregate. The percentage increase ranged between 4.22 to 5.17 %.

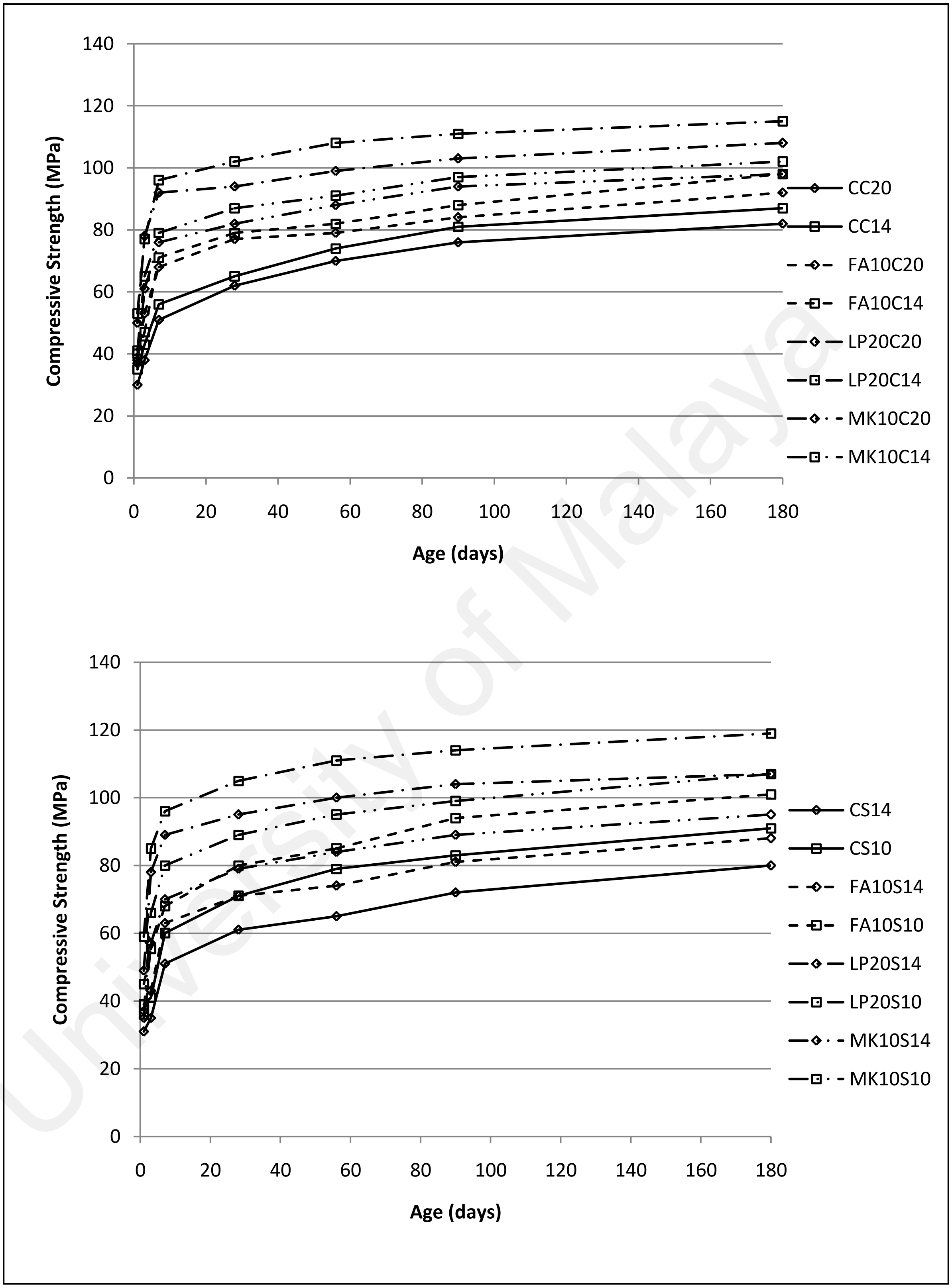


Figure 4.97: Compressive strength with different maximum size aggregate concretes

Table 4.12: Relative strength of mixes of different maximum size aggregate

Symbol of mix	Relative Strength	
	28d	180d
CC14/CC20	1.02	1.06
FA10C14/ FA10C20	1.03	1.07
LP20C14/ LP20C20	1.09	1.06
MK10C14/ MK10C20	1.06	1.04
CS10/CS14	1.16	1.13
FA10S10/ FA10S14	1.13	1.15
LP20S10/ LP20S14	1.11	1.11
MK10S10/ MK10S14	1.13	1.13

The effect of maximum size coarse aggregate on the tensile strength is presented in Figures 4.98. The results in Figure 4.98 indicate the comparison between the tensile strength of concrete mixes with 14mm and 20mm maximum aggregate size. All mixes with 14mm graded coarse aggregate exhibit tensile strength higher than those with 20mm graded coarse aggregate. Moreover, Figure 4.98 clearly reveals that concrete mixes with 10mm single size aggregate gives a splitting tensile strength higher than those with 14mm single size coarse aggregate. This is mainly attributed to the fact that mixes with 20mm maximum size have a specific surface lower and hence, the bond strength over the aggregate-cement interface is lower. This fact explains why the cracks and microcracks in the concrete sample follow the surface of larger particles. Yan et al. (2001) concluded that at the same mix proportion, the tensile strength increases with decreasing maximum size. Akcaoglu et al. (2004) concluded that in high strength concrete, the required stress levels to propagate crack decrease with increase in maximum size aggregate thus the tensile strength would reduce.

The effect of maximum size coarse aggregate on the behaviour of concrete in flexural tension status is presented in Figure 4.99. The mixes with 14mm continuous size coarse aggregate exhibit higher strength when compared with the mixes with 20mm continuous grade coarse aggregate as shown in Figure 4.99. The percentage increase at 28 days ranges between 1.5 to 3 %. Moreover, the mixes which have 14mm single size coarse aggregate gave lower flexural strength by approximately 4.9 to 6.1% than the mixes which have 10mm single sized coarse aggregate as shown in Figure 4.99. The fracture process in the mixes which contain small size coarse aggregate in flexural state is more difficult when compared with the mixes which contain large particles. In addition to the high strength of interfacial zone in high strength mixes, another reason is that during failure, the crack path is longer and more tortuous in the mixes which contained small particles due to the increasing number of particles.

Khaleel et al. (2011) studied the effect of type and maximum size of coarse aggregate on self compacting concrete properties at the same mix proportions and w/c ratio of 0.34. This work made comparison between 10mm and 20mm maximum size. The results showed that the decrease in maximum size led to increase in flexural tensile strength of mixes of different types of aggregate. Singh (1958) reported that for the same mix proportions, the use of smaller maximum size aggregate lead to increase in compressive and flexural strength compared to larger size.

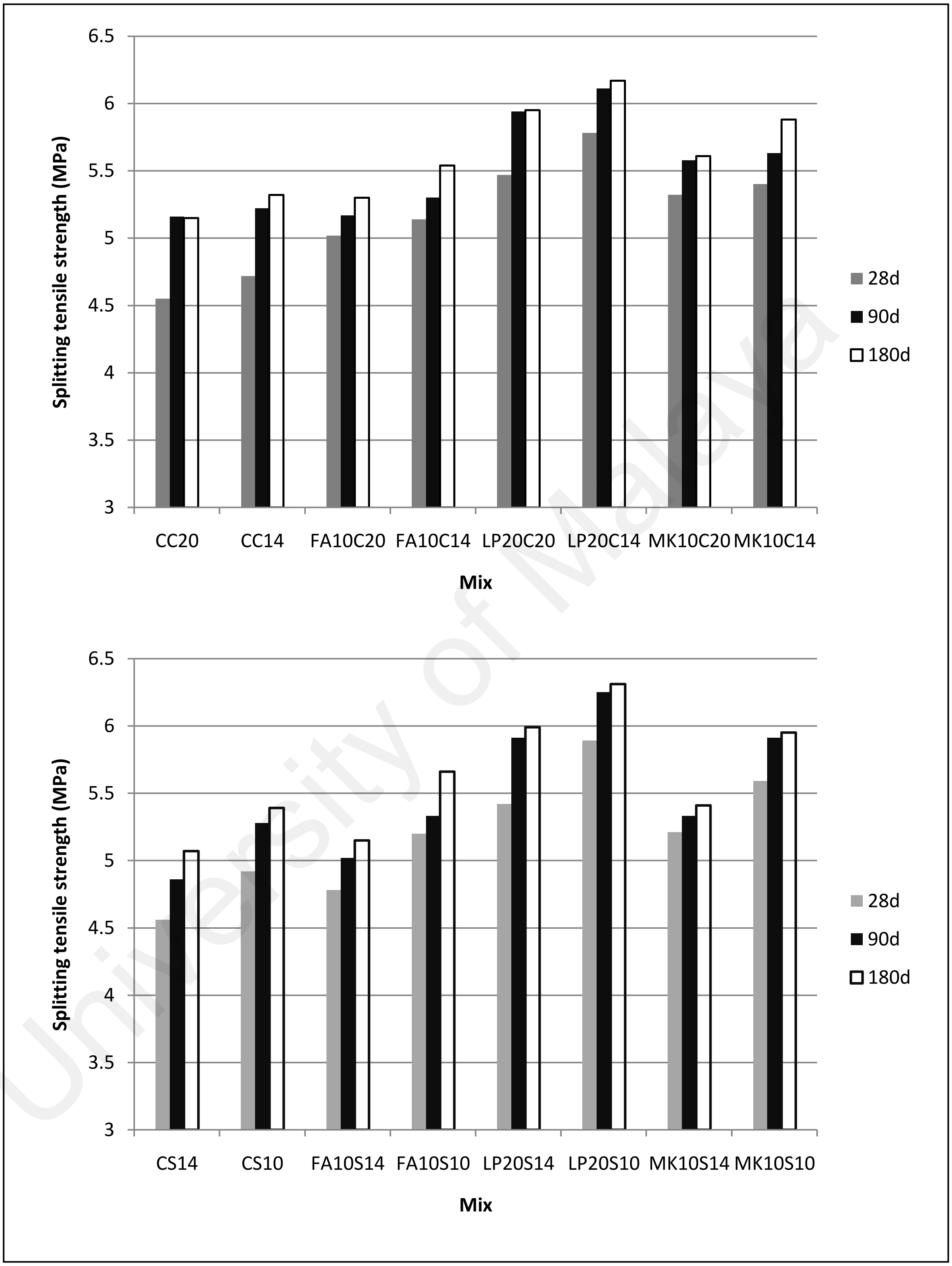


Figure 4.98: Splitting tensile strength of concretes with different maximum size aggregate

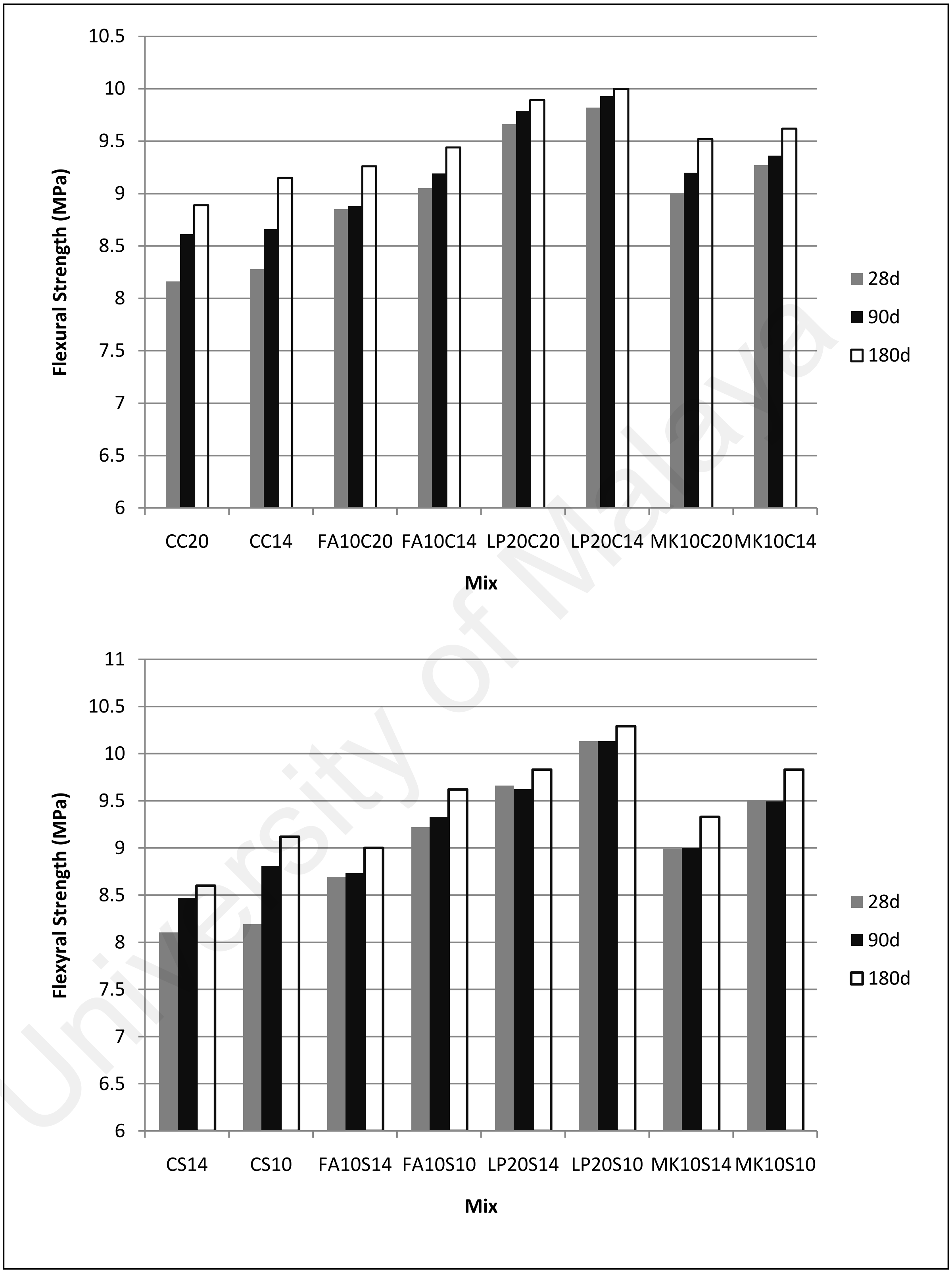


Figure 4.99: Flexural strength of concretes with different maximum size aggregate

Figure 4.100 shows the effect of maximum size of coarse aggregate on modulus of elasticity of mixes. Indeed, it appears from the Figure 4.100 that the smaller maximum size C14 has a larger modulus of elasticity compared with the larger maximum size C20 for all mixes. The percentage increase between these types of concrete mixes ranges between 0.2 to 2.3 %. Moreover, Figure 4.100 indicates that the smaller maximum size S10 has a larger modulus of elasticity compared with the larger maximum size S14 for all mixes. The percentage increase between these types of concrete mixes ranges between 4.1 to 5.8 %. This is because smaller concrete has a larger compressive strength than concrete with larger maximum size. This agrees with the study carried out by Shetty (2001) and Khaleel et al. (2011). Rahim (2005) concluded that the SCC mixes containing 10mm maximum size aggregate exhibited higher modulus of elasticity than that containing 20mm maximum size at the same mix proportions and w/c ratio.

Figures 4.101 show the comparison between water absorption of mixes with different size coarse aggregates. However, there is a slight difference between water absorption of mixes with small size and large size coarse aggregate.. Helmuth (1994) showed that for a given water-binder ratio, the use of larger maximum size aggregate led to increase in permeability of concrete. Basheer et al. (2005) investigated the influence of aggregate size and grade on the durability of concrete. This study indicated that the mixes containing 10mm maximum exhibited better durability than that containing 20mm maximum size at the same mix proportions.

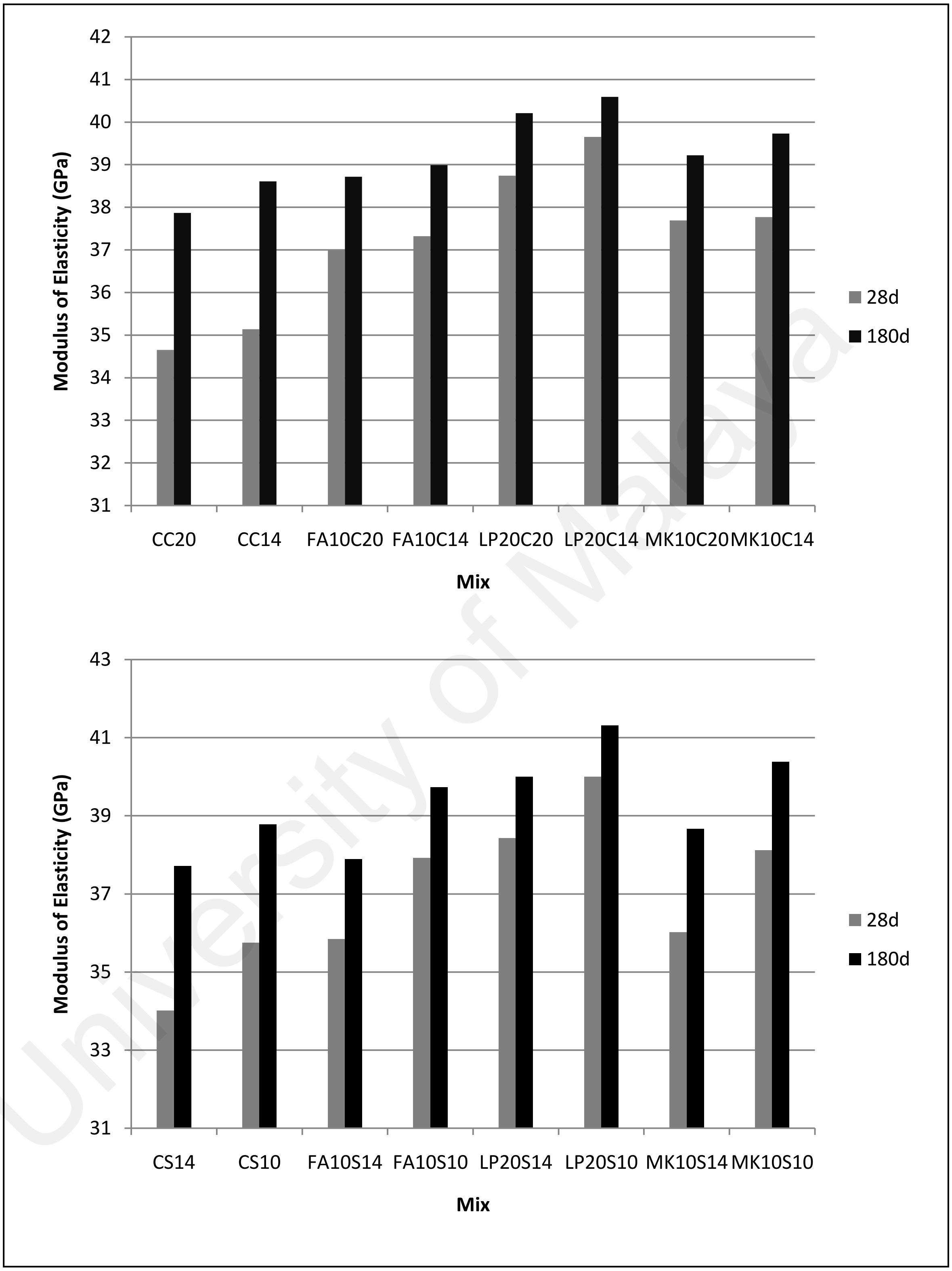


Figure 4.100: Modulus of elasticity of concretes with different maximum size aggregate

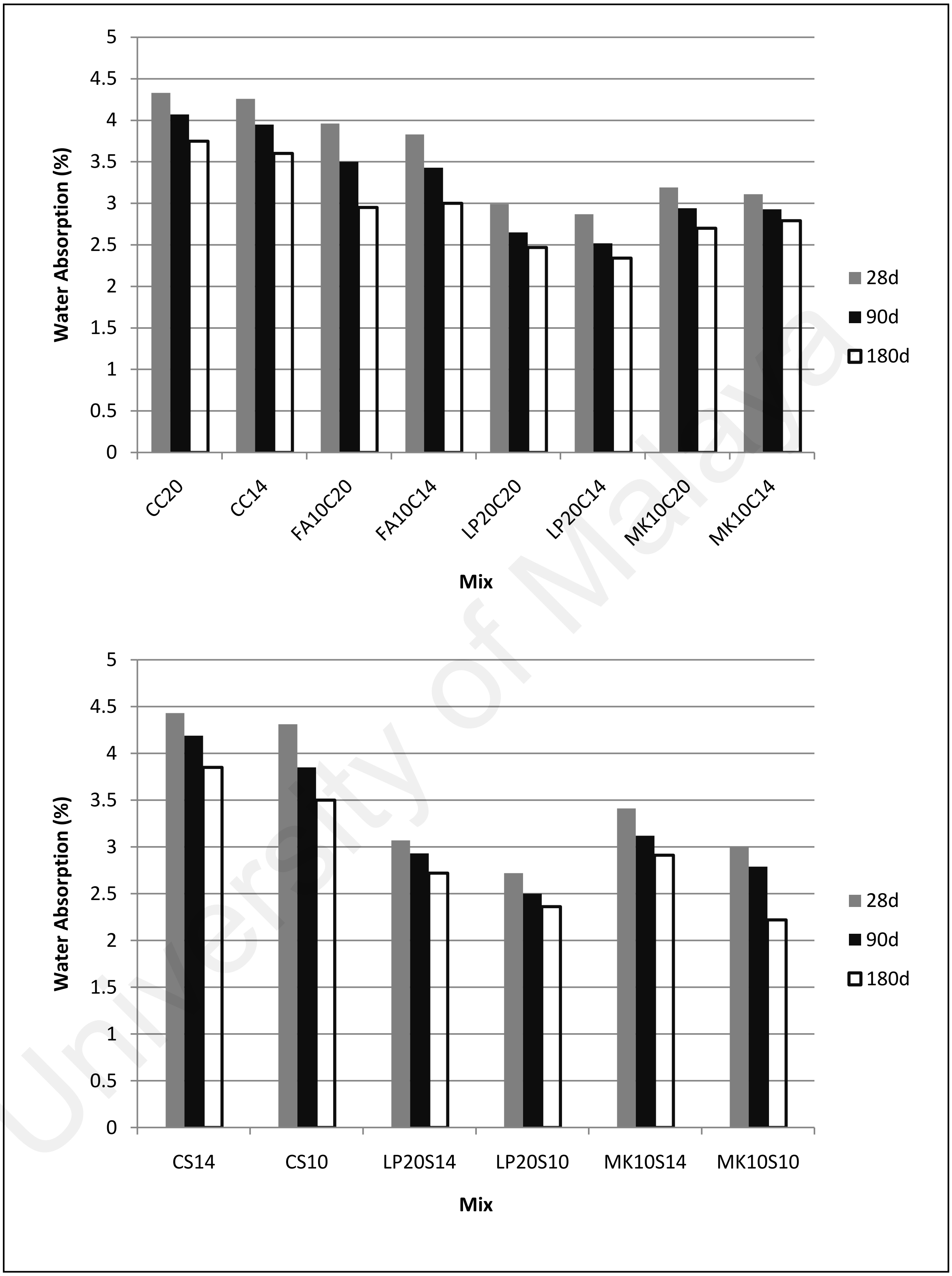


Figure 4.101: Water absorption of concretes with different maximum size aggregate

Maximum aggregate size is considered as an indirect factor which affects shrinkage of concrete. Indeed, the use of large maximum size has less surface area and requires less cement paste to cover particles to achieve the same workability as with small particles. In the present study, the mixes with different maximum aggregate size have the same amount of cement paste.

Figures 4.102 indicate that increasing the maximum size of coarse aggregate in concrete leads to a decrease in the drying shrinkage when compared to the small maximum size aggregate. This could be attributed to the fact that the larger size particle is better than the small size particle with regards to restricting the shrinkage of cement paste due to decreased surface area to volume ratio. This agrees with research carried out by Wash (1998a), Ozyildirim & Lane (2006), Shetty (2001), and Youjun et al. (2002).

Rahim (2005) studied the effect of maximum size aggregate on the shrinkage of concrete mixes with the same mix proportions. The results of this study showed that the mixes containing large aggregate particles reveal less shrinkage compared to smaller aggregate particles.

The results shown in Figure 4.103 represent the effect of maximum size of coarse aggregate on the UPV values. These values range between 4.72 to 5.71 km/sec and 5.41 to 6.03 km/sec at the 28 and 180 day mark respectively, thus improving the physical condition of the mixes. The ultrasonic pulse velocity when higher than 4.575 Km/sec reveals good quality of mixes (Safiuddin, 2008, Demirboga, et al. 2004). It can be clearly seen from these two figures that as maximum size of coarse aggregate increases, the UPV values decrease. This is compatible with the findings of Abo-Qudais (2005). He indicated that large maximum size aggregate in concrete lead to reduce wave velocity in the concrete sample. This study also indicated that the effect of water cement ratio on wave velocity was higher compared to the effect of maximum size aggregate. This study also showed that as water/cement ratio increase, the effect of maximum size aggregate on wave velocity is more significant.

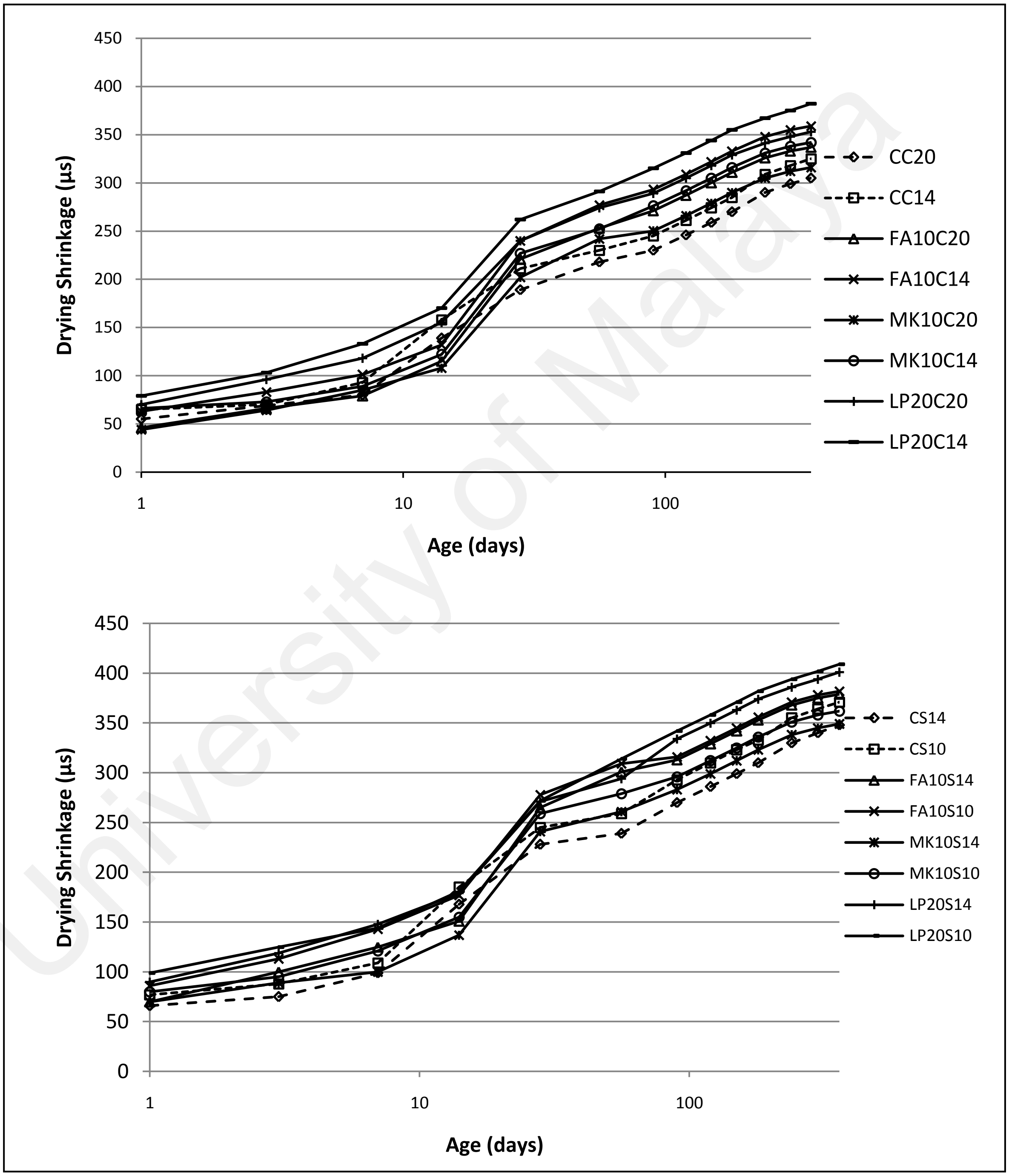


Figure 4.102: Drying shrinkage of concretes with different maximum size aggregate

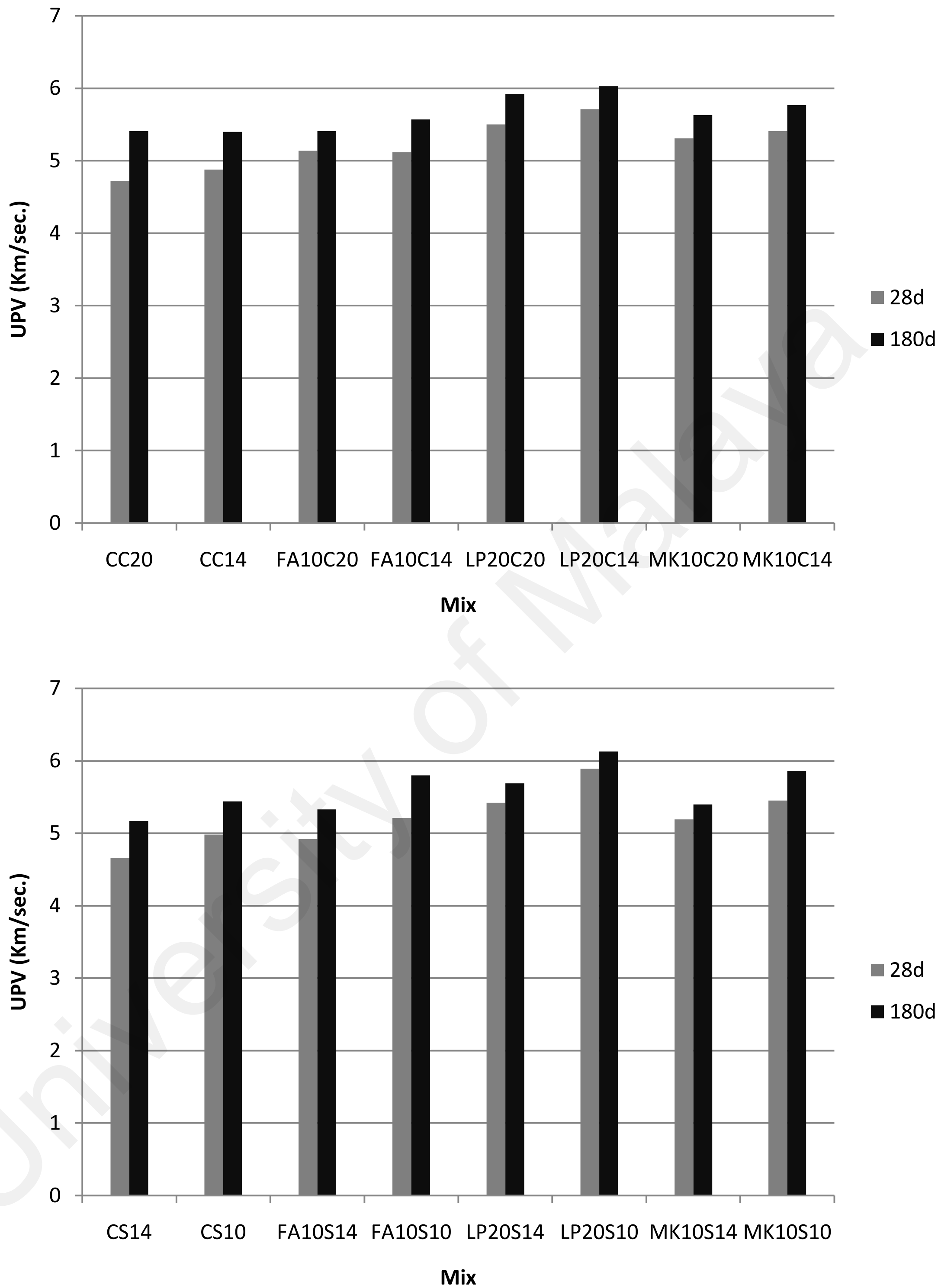


Figure 4.103: UPV values of concretes with different maximum size aggregate

4.7.4 Effect of coarse aggregate grading

Figure 4.104 shows the average compressive strength results at 1, 7, 28, 56, 90 and 180 days gained from cubes of the mixes with continuous increase in strength with age. Figure 4.104 indicates the effect of gap graded coarse aggregate on the strength of concrete. From the results presented in Figure 4.104, it is evident that the mixes with continuous grade aggregate a higher strength when compared with the mixes with gap graded aggregate. Indeed, the percentage increase of mixes with 20mm continuous grade coarse aggregate ranges between 4.8 to 5.6% compared with 20mm gap grade at 28 days. Moreover, the results shown in Figure 4.104 reveal that the use of 14mm single size aggregate in the mixes reduces the compressive strength by approximately 7 to 11.5% when compared with the use of 14mm continuous aggregate in the concrete mixes at 28 days. The percentage difference between single sized aggregate and continuous grade is more than the percentage difference between gap graded aggregate and continuous grade of coarse aggregate. This could be due to the fact that the voids ratio in cases where single sized coarse aggregate was used is more than the voids ratio in case of using gap grade of coarse aggregate in the mixes.

The reduced strength by using gap graded coarse aggregate in mixes is due to the fact that it leads to a reduction in the interaction between the intermediate sizes particles. However, the single size aggregate lacks fine particles smaller than 10mm and 4.75mm which in turn would lead to reduction in the packing density and thus a reduction in concrete strength. This is in agreement with results of Kwan and Mora (2001), Shilstone (1990), and Cramer and Hall (1995). Meddah et al. (2010) show that continuous size distribution yields mixes with the highest compaction. In other words, the high compaction provides mixes with low void ratio and consequently increases the compressive strength.

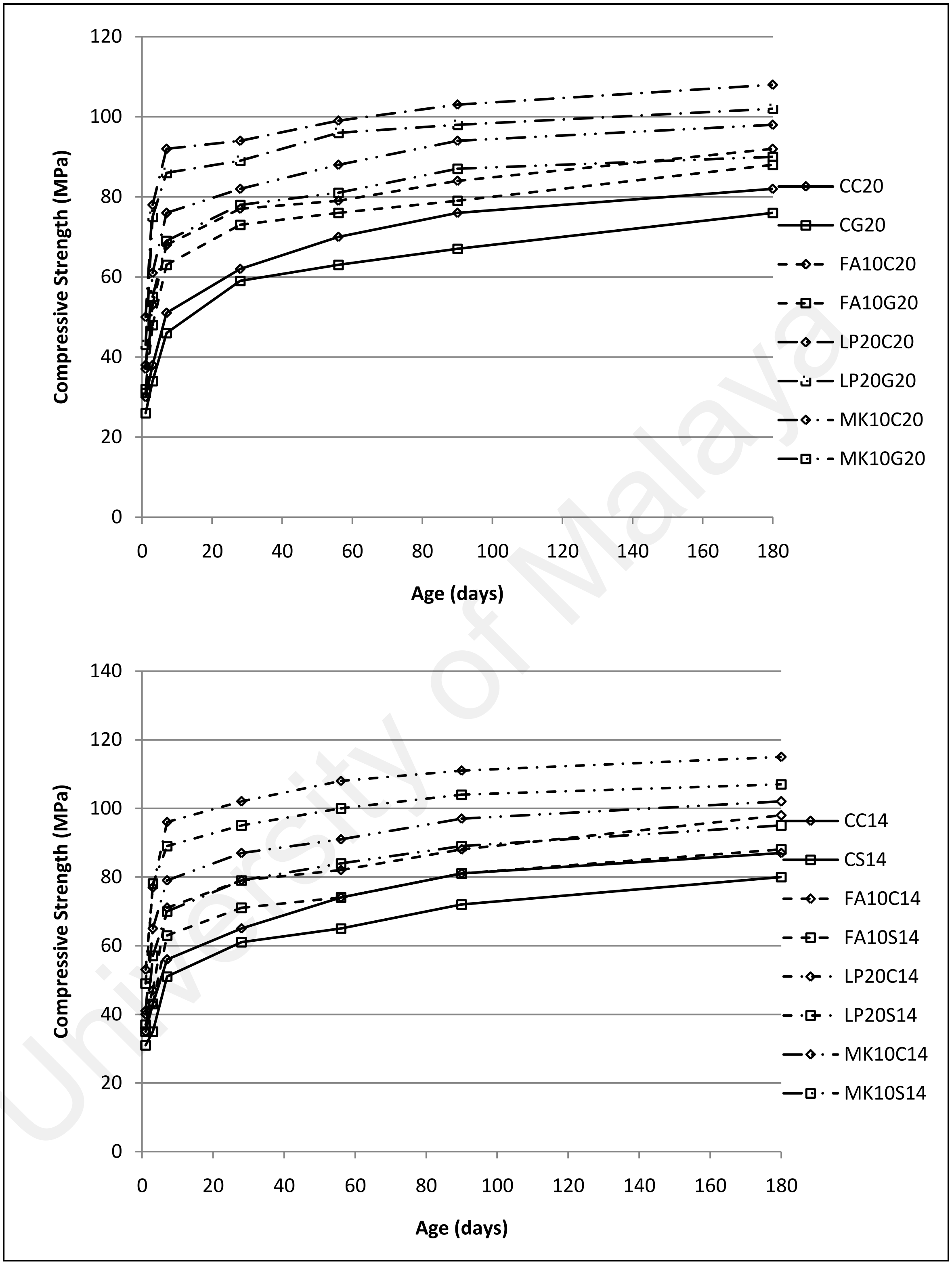


Figure 4.104: Compressive strength of concretes with different grade aggregate

The results presented in Figure 4.105 illustrate that the use of gap graded coarse aggregate weakens the tensile strength of concrete mixes when compared with the mixes which contain continuous grade coarse aggregate. The percentage strength reduction in the mixes which contain gap graded coarse aggregate varies between 2.24 to 4.8% compared with the mixes which contain continuous aggregate. The packing density of mixes which contain gap graded aggregate is less than the packing density of mixes with continuous aggregate whilst low packing density refers to high voids ratio. In other words, as voids ratio increases the weak positions also increase thus the strength is reduced. In addition, Figure 4.105 also reveals that the use of single size aggregate in the mixes leads to a reduction in tensile strength when compared with the mixes which contain coarse aggregate with continuous grade.

The reason for this is similar to that mentioned above, where the concrete mixes which have single size aggregate display less packing density and thus higher voids ratios exist when compared with mixes with continuous grade aggregate. Therefore, the tensile strength is less (Wong and Kwan, 2008).

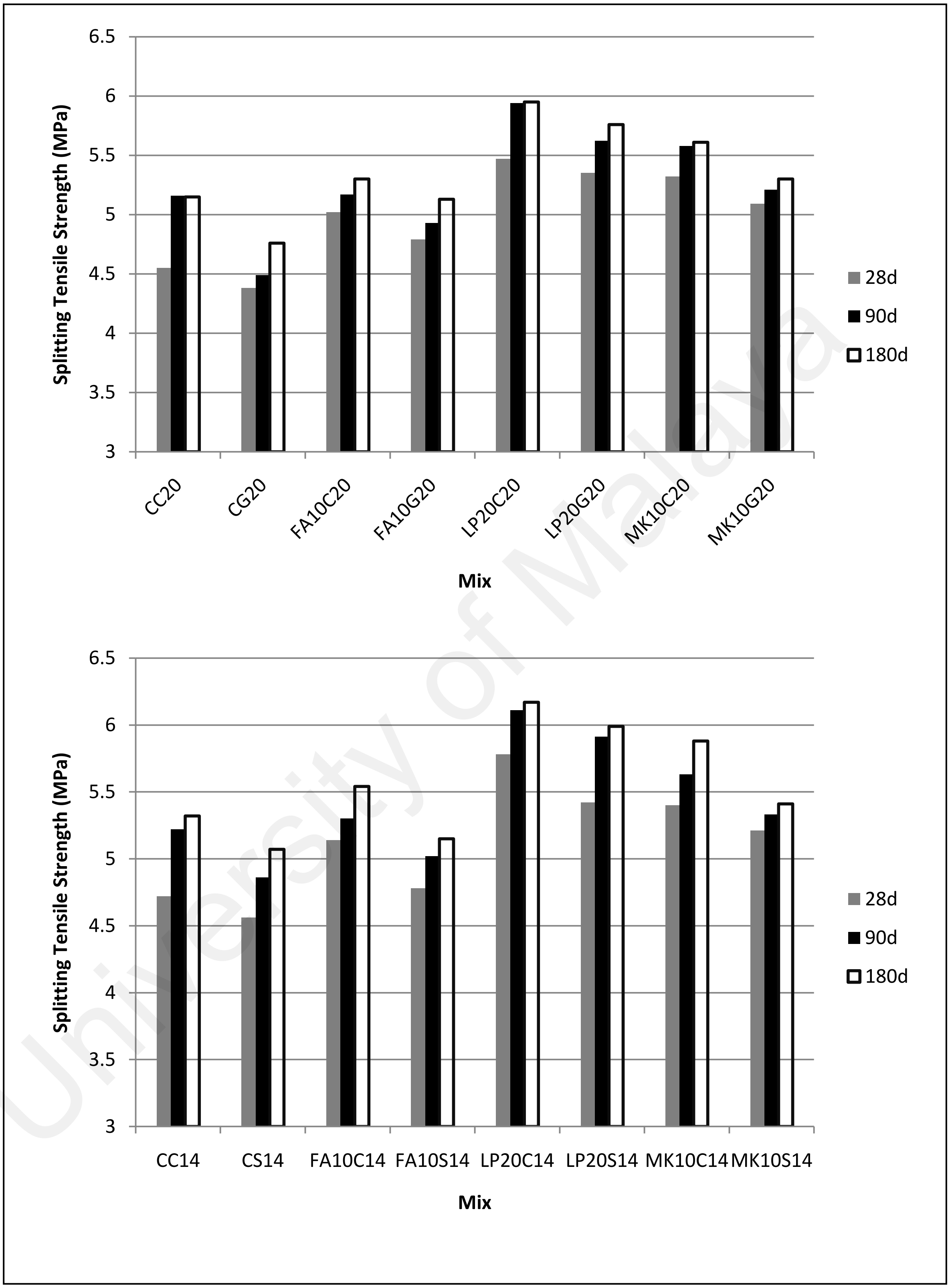


Figure 4.105: Splitting tensile strength of concretes with different grade aggregate

The effect of grading of aggregate on flexural strength is shown in Figure 4.106. Figure 4.106 shows that the concrete mixes prepared with gap graded size of coarse aggregate give flexural strength values lower than concrete mixes prepared with continuous grade. The percentage reduction in flexural strength of concrete mixes with gap graded coarse aggregate relative to their concrete mixes with continuous grade are 2.33 to 5 % for all mixes. In addition, Figure 4.106 illustrates that the use of single sized coarse aggregate in the mixes gives low values of flexural strength when compared with the use of coarse aggregate with continuous grading. The use of gap graded or single sized coarse aggregate in the mixes reduces flexural strength due to firstly, the reduction occurring in the specific surface area which in turn reduces the interface zone between aggregate and matrix and thus reduces the strength. Secondly, the voids ratio in mixes such as these is high when compared to the mixes with continuous grade aggregate.

Figure 4.107 illustrates the effect of the use of gap graded coarse aggregate on the modulus of elasticity of concrete. The results show that the mixes with gap grade coarse aggregate have low modulus of elasticity when compared with the mixes with continuous grade of coarse aggregate. The modulus of elasticity of concrete is affected by the voids ratio in concrete. As mentioned above, the mixes with gap graded coarse aggregate have higher voids when compared with the mixes with continuous grade coarse aggregate which in turn reduces modulus of elasticity of mixes.

Figure 4.107 indicates that concrete mixes with single sized coarse aggregate leads to a slight reduction in modulus of elasticity when compared with concrete mixes with continuous grade coarse aggregate.

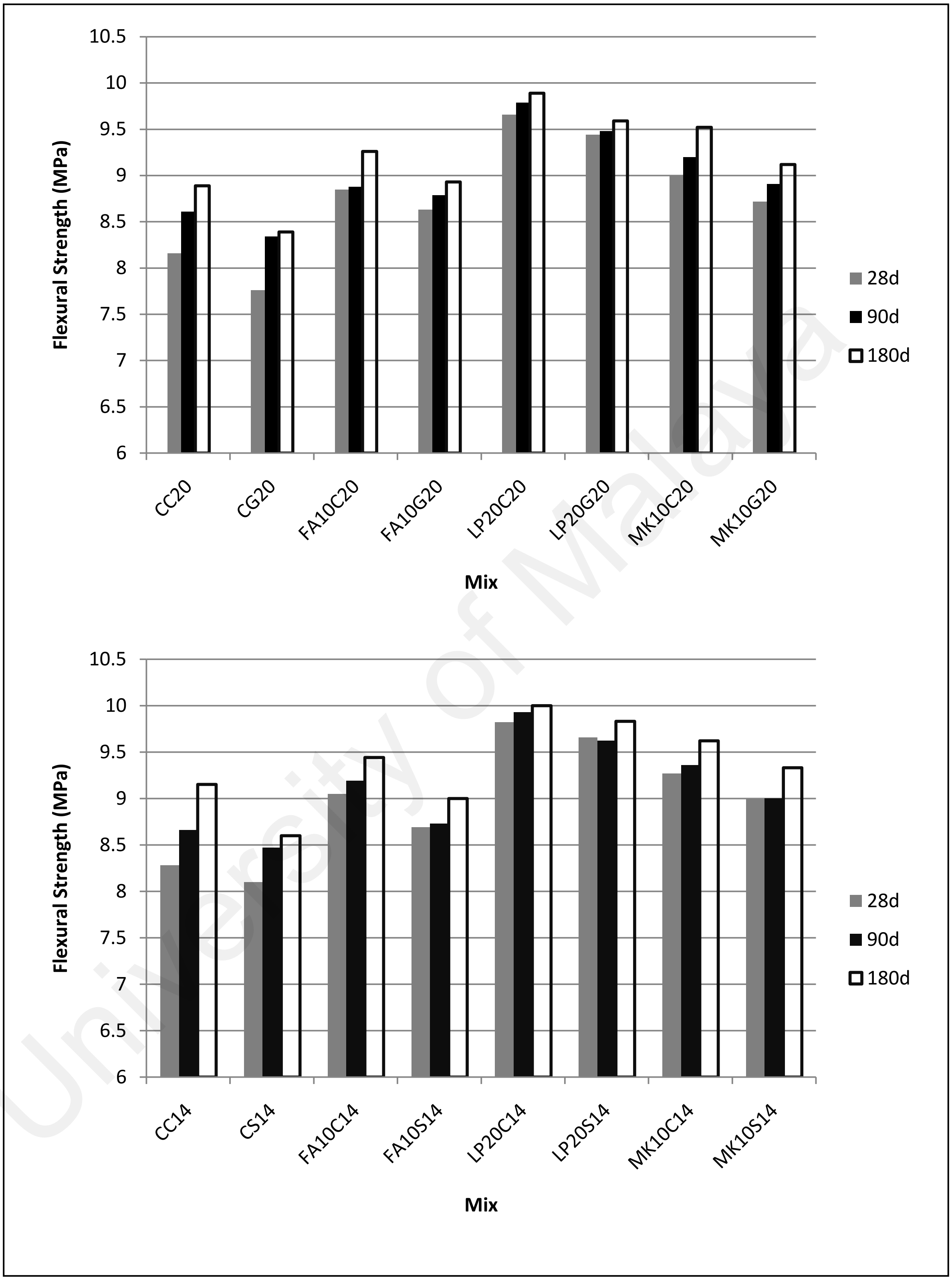


Figure 4.106: Flexural strength of concretes with different graded aggregate

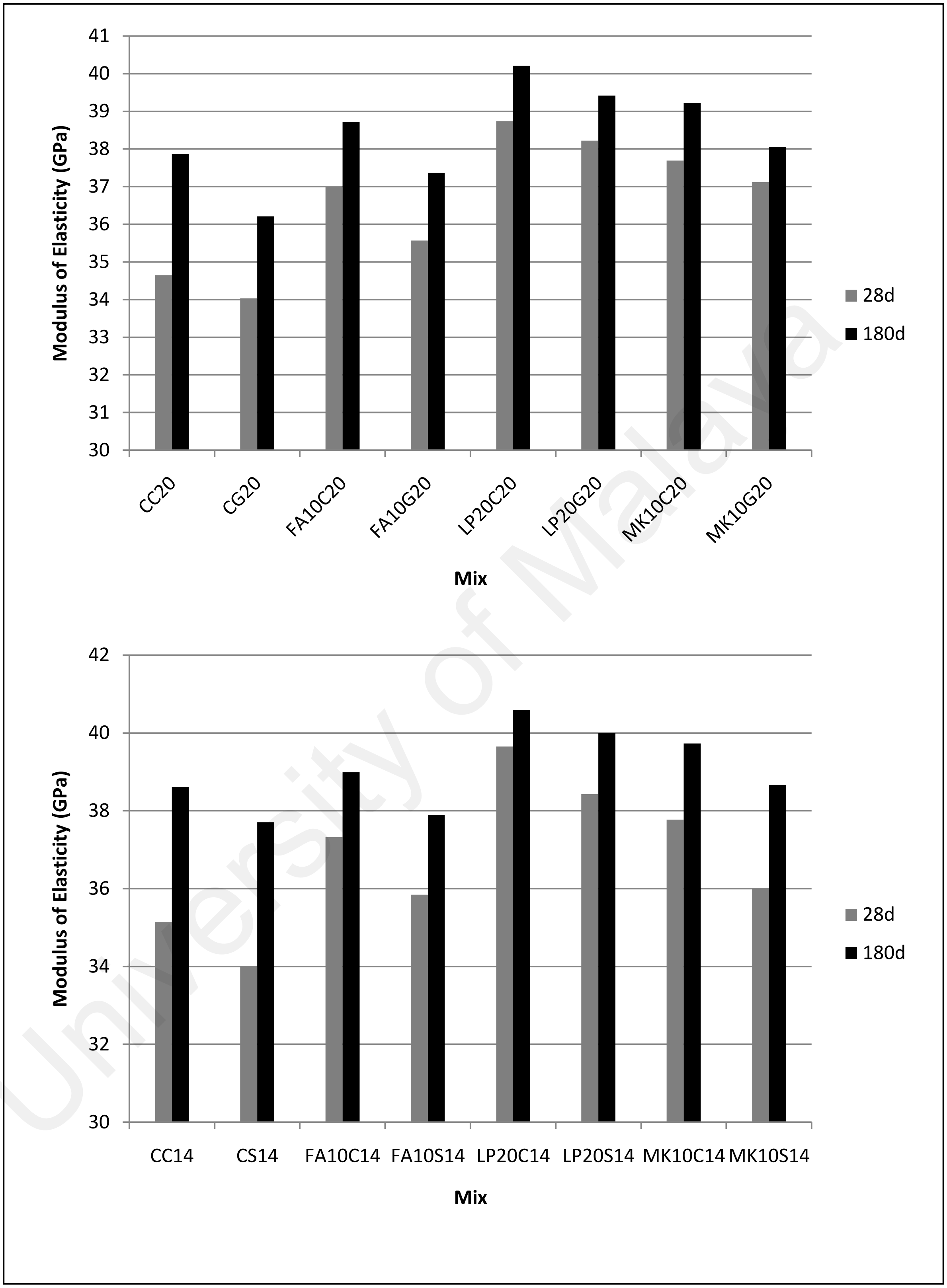


Figure 4.107: Modulus of elasticity of concretes with different grade aggregate

Water absorption with three types of grading is presented in Figure 4.108. As is evident from these figures, the water absorption of mixes which contain gap grade or single sized coarse aggregate is higher than the water absorption of mixes with continuous grade coarse aggregate. This results from the fact that high voids ratio in the mixes which contain gap grade or single sized coarse aggregate is higher and thus leads to an increase in the permeability of the mixes and an increase in their water absorption.

The grading of aggregate has a significant role in affecting some properties of concrete in fresh or hardened state. One of these characteristics is dimensional stability of concrete. In this study, three types of grading are used; continuous grade, gap grade and single size grading. Uniformly distributed mixtures generally lead to a higher packing which results in concrete with higher density and less permeability (Golterman et al., 1997; Glavind et al,, 1993; Johansen and Andersen, 1989).

The experimental results are represented in Figures 4.109. Figure 4.109 indicates that the use of gap graded aggregate leads to higher shrinkage when compared with continuous grade at all ages. On the other hand, Figure 4.109 indicates that the mixes which have single size coarse aggregate lead to an increase in drying shrinkage values for mixes with all single size when compared with the mixes which have coarse aggregate with continuous grade. This may be due to the fact that the mixes which have single size coarse aggregate have high voids ratio and thus higher shrinkage. This agrees with Khaleel et al. (2011).

This increment in shrinkage is due to the fact that nonuniform grade mixes need to cement mortar more than that of uniform grade mixes in order to fill all of the voids. Since the cement mortar of all mixes is identical, the void ratio in gap graded mixes is more than that in continuous grade mixes.

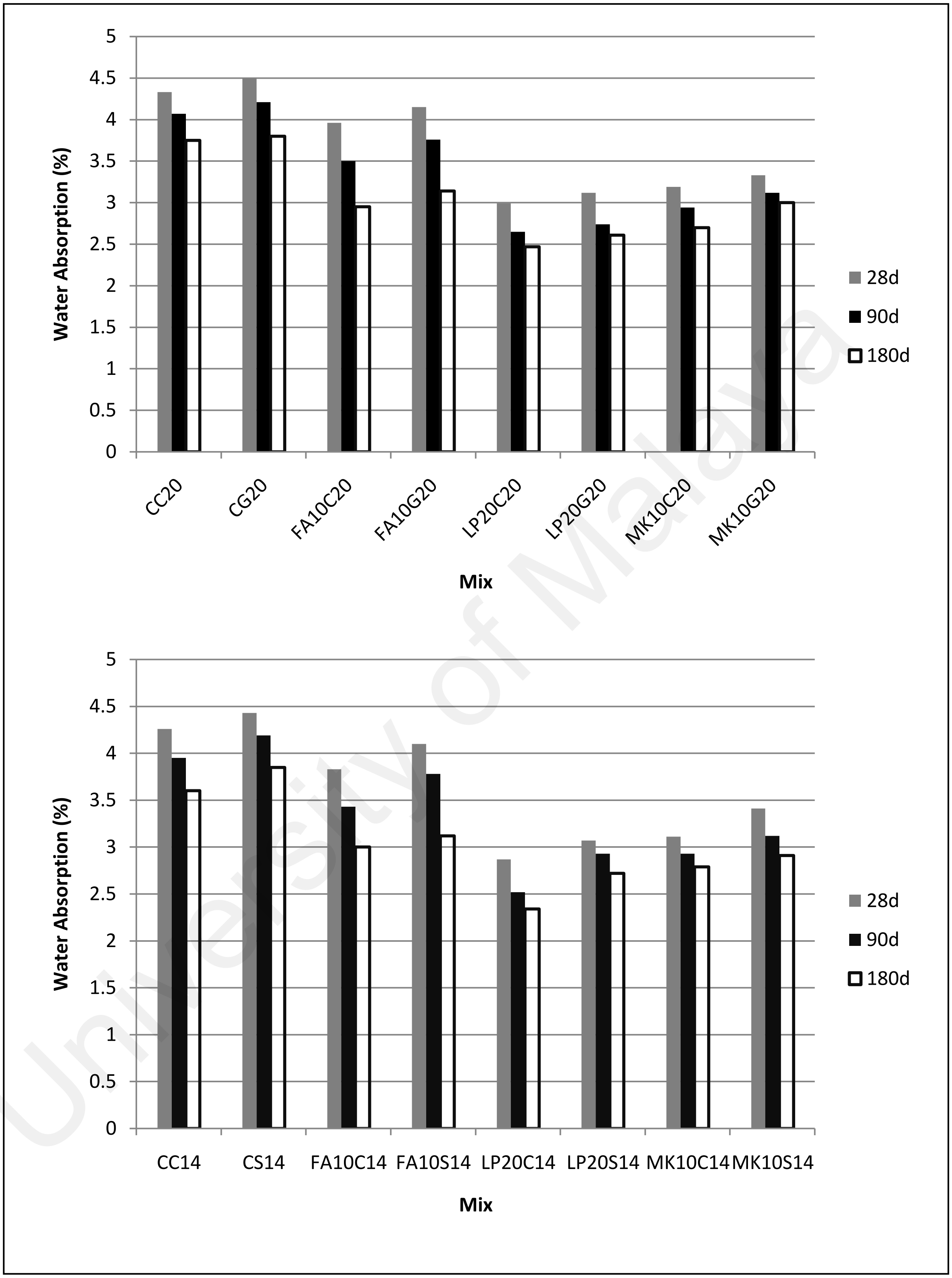


Figure 4.108: Water absorption of concretes with different graded aggregate

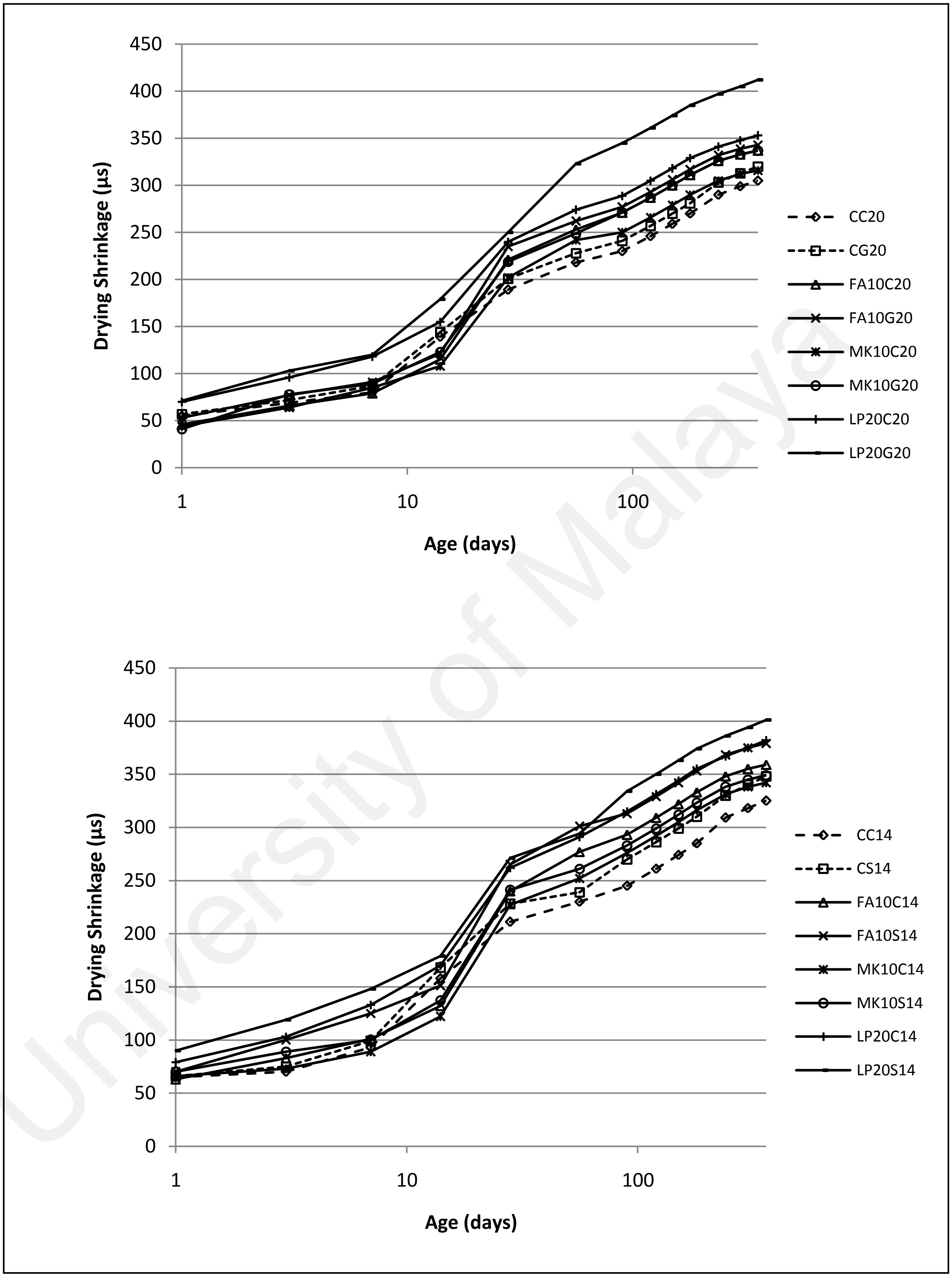


Figure 4.109: Drying shrinkage of concretes with different graded aggregate

The results of ultrasonic pulse velocity measurements for graded and irregular grade aggregate mixes at ages of 28 and 180 days are plotted in Figures 4.110. It is obvious that values of ultrasonic pulse velocity increase with age of specimen. This is mainly due to the increase in the density of the specimen with age and the reduction in points of discontinuity.

It can be seen from Figure 4.110 that gap graded coarse aggregate concretes have low values of ultrasonic pulse velocity when compared with continuous grade coarse aggregate concretes. Moreover, Figure 4.110 shows that the employment of single sized coarse aggregate in mixes gives ultrasonic pulse velocity values lower than employment of coarse aggregate with continuous grade. This is attributed to the fact that the mixes which contain a single coarse aggregate have a higher void ratio when compared with mixes which contain a continuous grade coarse aggregate thus leading to less pulse velocity.

Graded aggregate have a good packing density compared to single size aggregate. Where in the case of graded aggregate, the small particles fill the gaps between large particles and thus increase packing density and this is as opposed to single size aggregate which has high voids (Lamond and Pielert, 2006). Graded aggregate has low void ratio and thus would require less mortar or paste and this would lead to decrease segregation, bleeding, shrinkage, creep and would produce concrete which is more durable (Quiroga and Fowler, 2004)

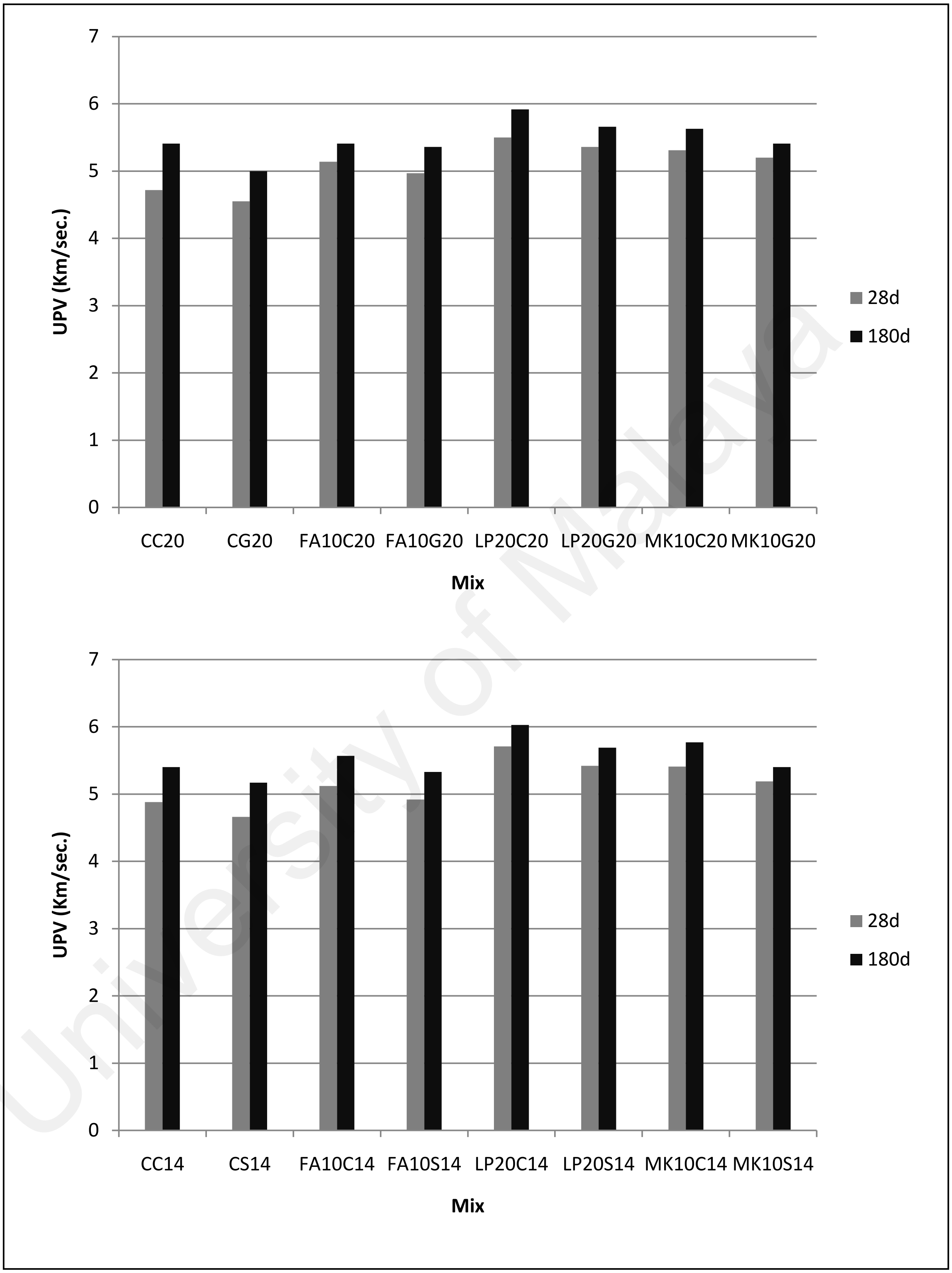


Figure 4.110: UPV values of concretes with different graded aggregate

4.7.5 Effect of coarse aggregate flakiness

The results in Figure 4.111 show the effect of the employment of coarse aggregate particles with a high flakiness ratio on the compressive strength of concretes. As the flakiness ratio in aggregate particles increases the compressive strength is reduced. For example, at 28 days the value of the compressive strength of CSHFR10 and FA10S10HF is 47 and 59 MPa respectively, while the value of compressive strength of CS10 and FA10S10 is 71 and 80 MPa respectively. The increase in the flakiness ratio in aggregate particles leads to a reduction in the compactibility of mixes which thus increases the voids in turn and weakens the strength. This is compatible with results of Mukhopadhyay (2004) and Chen et al. (2005). Shilstone (1990) mentioned that high ratios of elongated and flaky particles exist in sizes between 9.5mm to 2.36mm of coarse aggregate. Galloway (1994) mentioned that the flat particles weaken the strength and durability due to that the flat particle may be oriented someway so that it could impair strength.

Chen et al. (2005) concluded that flaky particles exhibited to higher breakage probability especially during the mixing and compaction and have less compactibility in turn would affect badly in strength.

Figure 4.112 shows the effect of the use of flaky particles in the coarse aggregate on the tensile strength of mixes. The results reveal that the employment of high flakiness ratio aggregate reduces the tensile strength of mixes. The use of flaky aggregate particles increases the gathering of the bleeding water under the particles, and this weakens the bond strength of the transition zone which in turn reduces the strength. Chen et al. (2005) concluded that the use of aggregate with higher flaky particle ratios lead to decrease in the tensile strength of asphalt concrete.

The flexural strength values of the mixes which contain different ratios of flaky aggregate particles are presented in Figure 4.113. The represented results show that the aggregates with a high ratio of flaky particles lead to a decrease in the flexural strength of the mixes. It is well known that the flexural strength is affected by transition zone between matrix and coarse aggregate whilst flaky particles confine the bleeding water underneath it and thus weaken the flexural strength. The increased flaky particles need a great deal of matrix to fill the voids. At the same matrix content, the mixes with a high ratio of flaky particles have low packing density when compared with the mixes with a lower ratio of flaky particles.

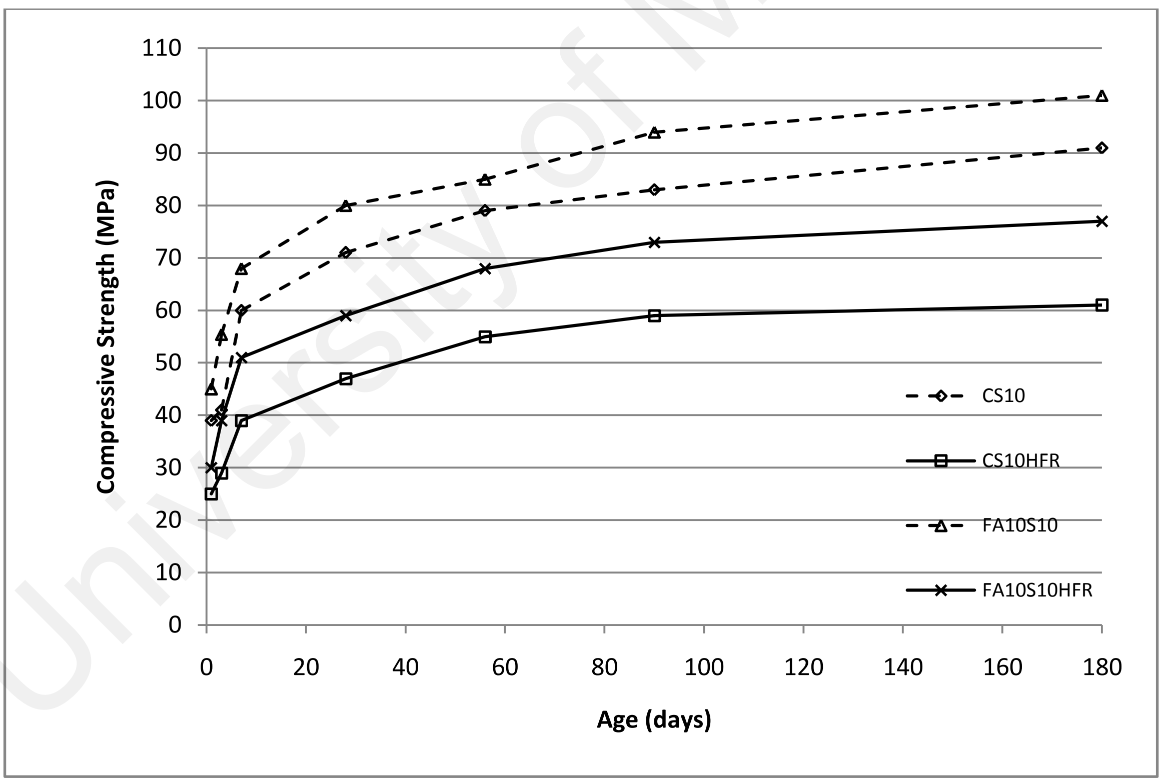


Figure 111: Compressive strength of concretes with different shape aggregate

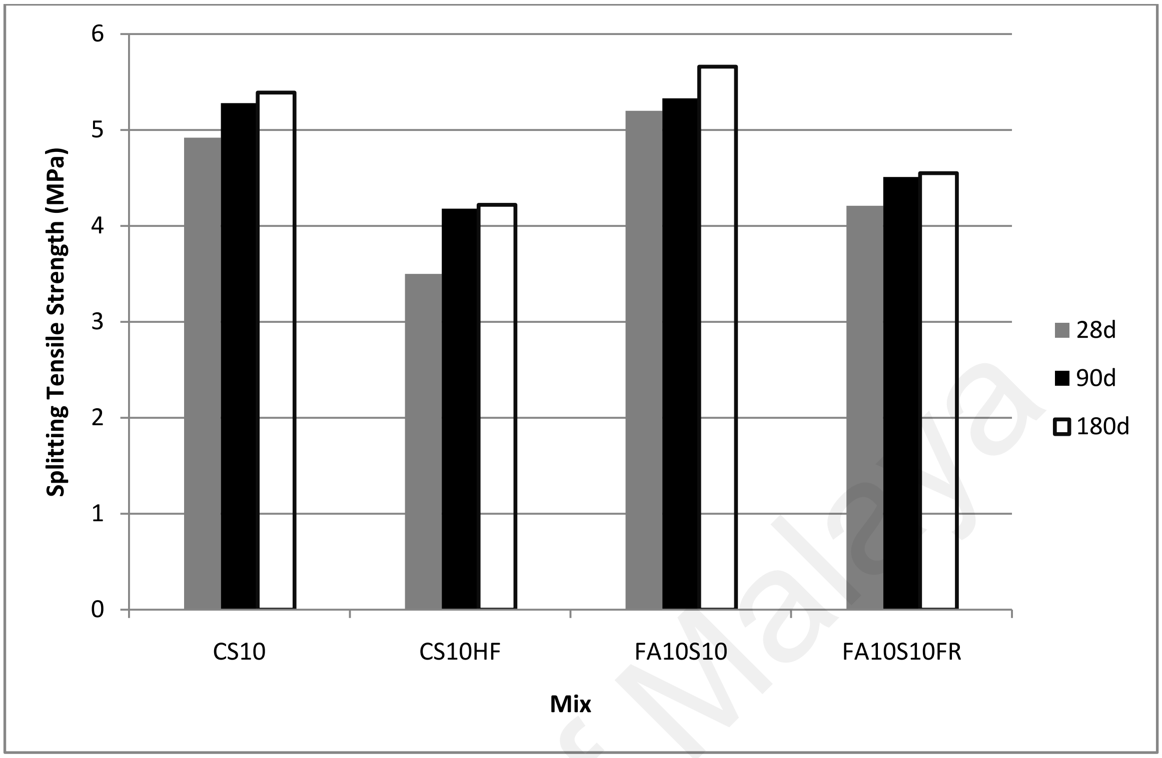


Figure 4.112: Splitting tensile strength of concretes with different shape aggregate

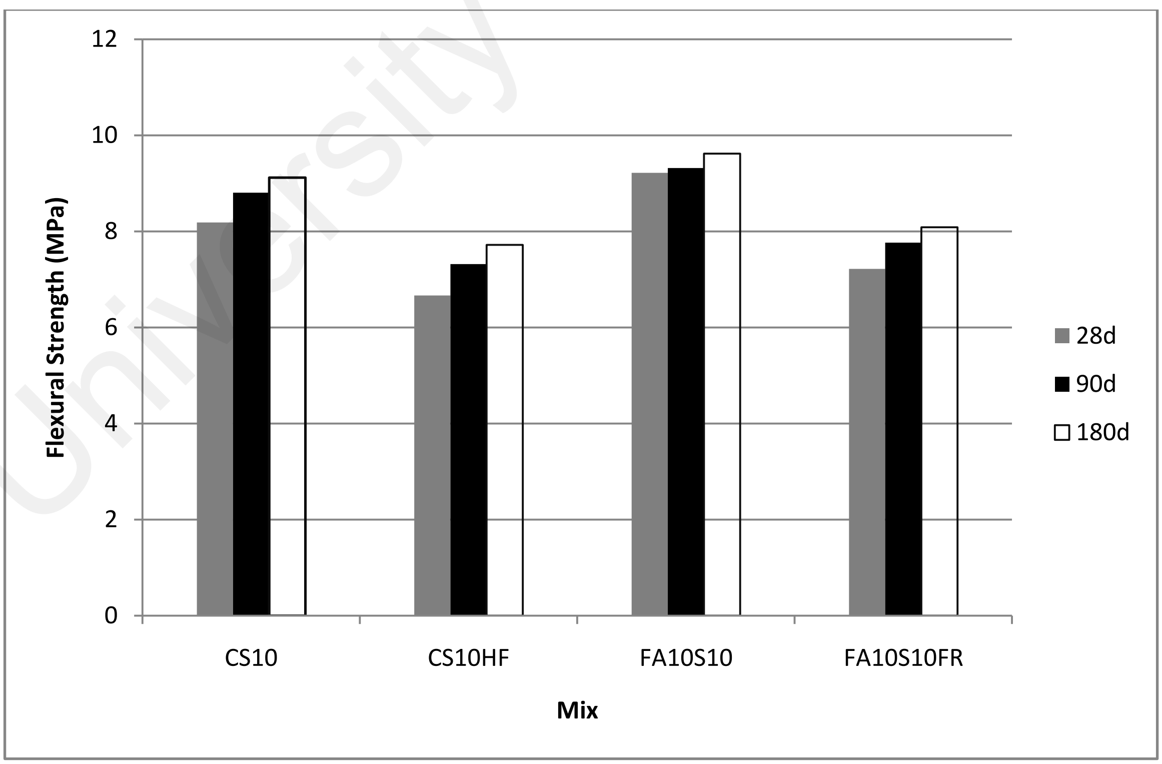


Figure 4.113: Flexural strength of concretes with different shape aggregate

Figure 4.114 shows that the modulus of elasticity of CS10HFR and FA10S10HFR mixes exhibit low values when compared with the modulus of elasticity of CS10 and FA10S10 respectively. The aggregate with high amount of flaky particles reduces the compactibility of concretes as well as confining the bleeding water under it and thus increasing the voids and reducing the bond strength of the interfacial transition zone. Siswosoebrotho et al. (2005) concluded that the resilient modulus of asphalt concrete decrease with increasing flakiness index of coarse aggregate.

Figure 4.115 indicates the effect of the presence of coarse aggregate particles in concrete with a high flakiness ratio on the water absorption of mixes. The use of aggregate with a high volume flaky particles reduces the compactibilty of concrete when compared with other mixes thus increasing the voids ratios. In addition to this, high flaky particles confine bleeding water. The evaporation of bleeding water leaves large voids which assist with increasing the water absorption of mixes.

According to Mehta and Montoero (2006), the absence of compactibilty increases the voids. Kosmatk (1994) studied the effect of flat particles on concrete pavements. This study indicated that flat particles which are closer to the surface of the concrete prevent from the arrival of water to mortar above particle leading to deterioration of the concrete surface.

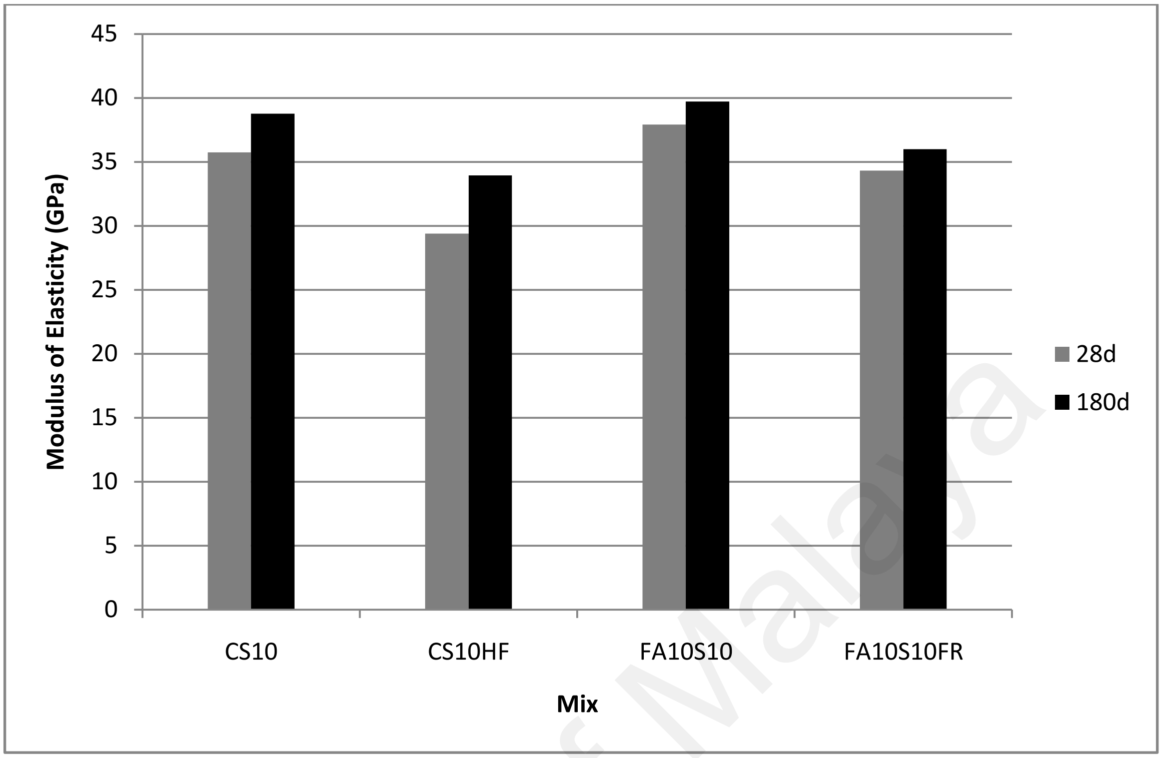


Figure 4.114: Modulus of elasticity of concretes with different shaped aggregate

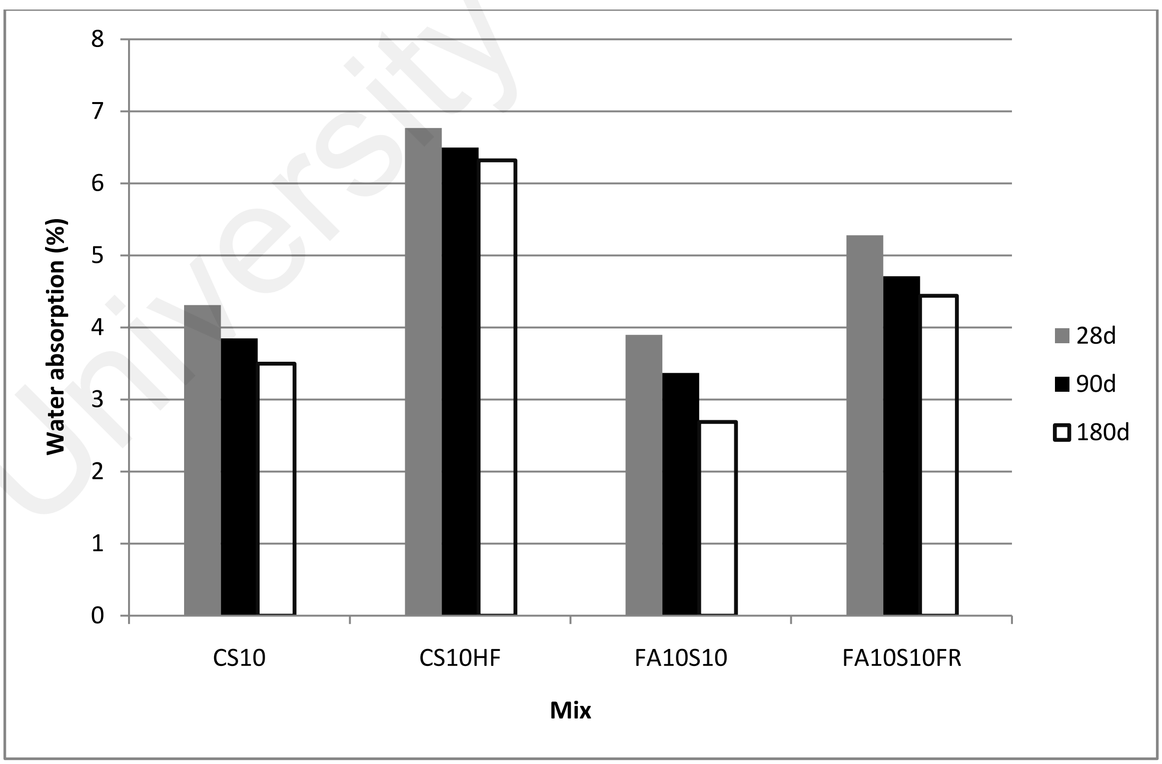


Figure 4.115: Water absorption of concretes with different shaped aggregate

The results show that the mixes containing high flakiness ratio of coarse aggregate particles shrink more than the mixes which contain low flakiness ratio as presented in Figure 4.116. High flakiness ratio of coarse aggregate particles means high bleeding of water and high void ratio in concrete. This leads to a great deal of water content evaporating which leads to increased drying shrinkage.

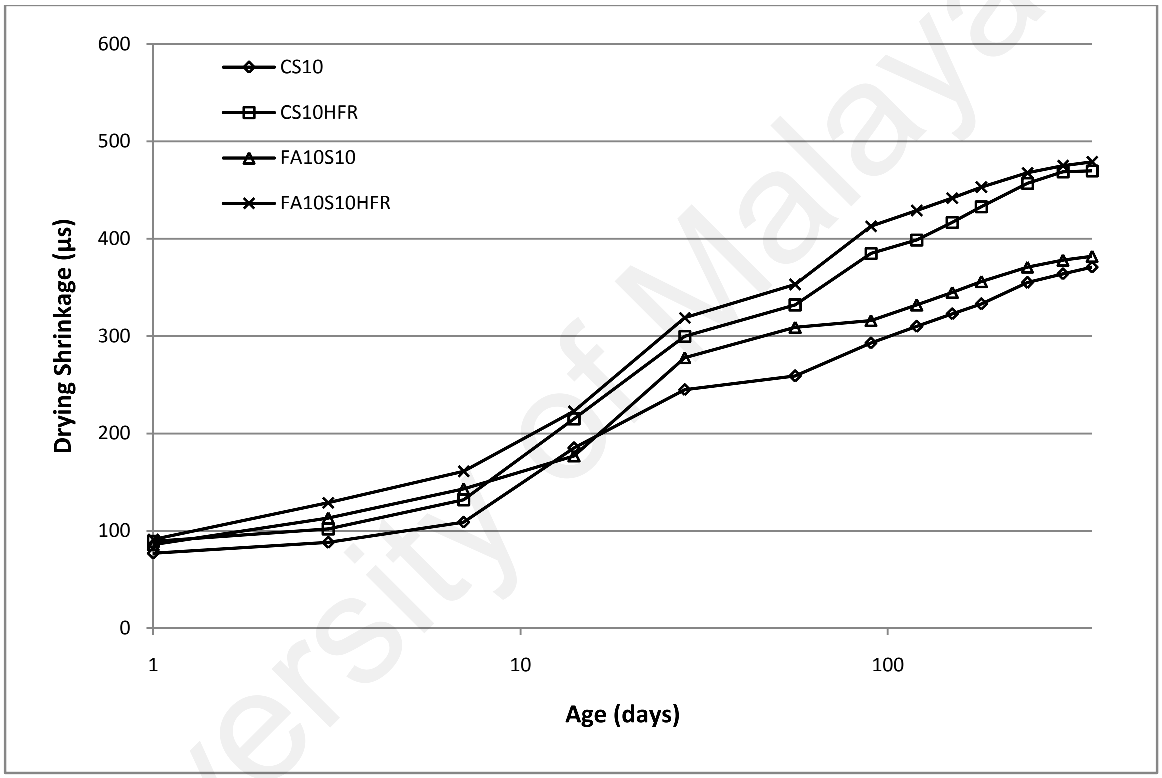


Figure 4.116: Drying shrinkage of concretes with different shaped aggregate

The effect of the presence of high amounts of flaky particles in the concrete mixes on UPV values is presented in Figure 4.117. The results show that increase in flaky particles in the mixes leads to a reduction in velocity of pulse due to the fact that the presence of this type of particle affects the compaction of mixes so that mixes with high voids content are produced and thus would affect strength and UPV results. In addition to this, the bleeding water of such mixes is high compared with other mixes. During the drying shrinkage, this bleeding water evaporates leaving high void in the mixes. Therefore, UPV values are less.

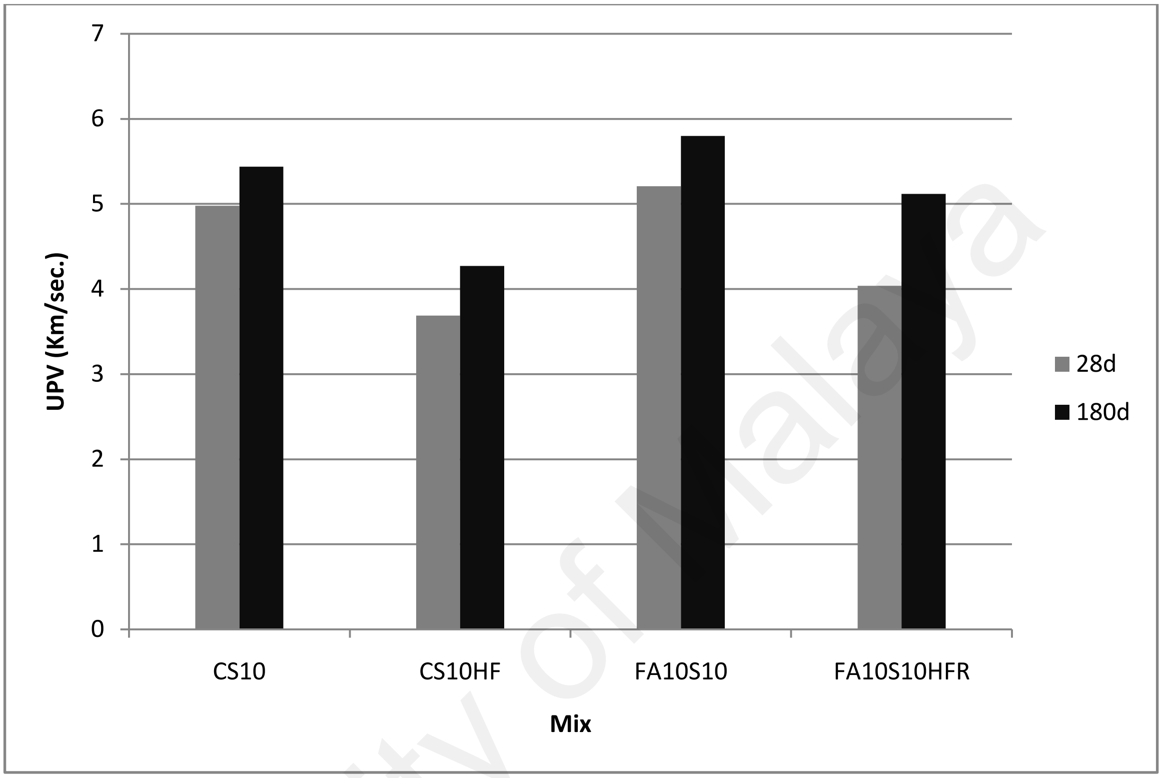


Figure 4.117: UPV of concretes with different shaped aggregate

4.8 Test result correlation

4.8.1 Flowabity of mortar and concrete

Through trial mixes mentioned in 3.3.3 (Table 3.6), relationships are drawn between the slump flow of concrete and mortar as shown in Figure 4.118. This relationship shows that as slump flow diameter of mortar increases the slump flow of concrete also increased. This relationship differs according to the mix proportion of mixtures. When coarse aggregate volume in concrete is 29%, the increase in flow ability of mortar leads to an increase in the flow ability of concrete with high degree and this increase will decrease as coarse aggregate volume is increased. This is attributed to the fact that at coarse aggregate volume of 29%, the mortar volume is high (71%) and the effect of increase in flow ability of mortar on the concrete is higher than other mixes. As mortar volume is increased, the mortar layer around the aggregate particles is thicker and thus increases the distances between particles which in turn reduces the collusion and thus improves the flow ability. Furthermore, according to the relation drawn in Figure 4.118 for various aggregate content (various mortar content) in concrete, and by interpolation a good relationship can be drawn between coarse aggregate volume and the mortar slump flow at concrete slump flow of 650mm as shown in Figure 4.119. This relationship gives correlation coefficient of the regression analysis (R^2) of 0.983 as shown in Figure 4.119. The following equation is deduced from this relationship:

VR % = 0.194(MSF) – 18.76 ……………………………………………….. **(4.1)**

Where:

VR: volume ratio of coarse aggregate in concrete
MSF: slump flow of mortar

From this equation, it can be estimated the coarse aggregate volume ratio in self compacting concrete to achieve the requirement of 650mm slump flow value.

4.8.2 Relationship between BR and ΔH

The blocking ratio in the L-box test and the difference in height in J-ring tests results are correlated with the polynomial relationship as shown in Figure 4.120. The correlation coefficient is 0.863. The good correlation results from the similar trends of variation of blocking ratio and different heights for all mixes. Indeed, the correlation shows that as blocking ratio decreases the different height increases. This is because the passing process through steel bars suffers from the same problem, namely the blocked aggregate behind the bars.

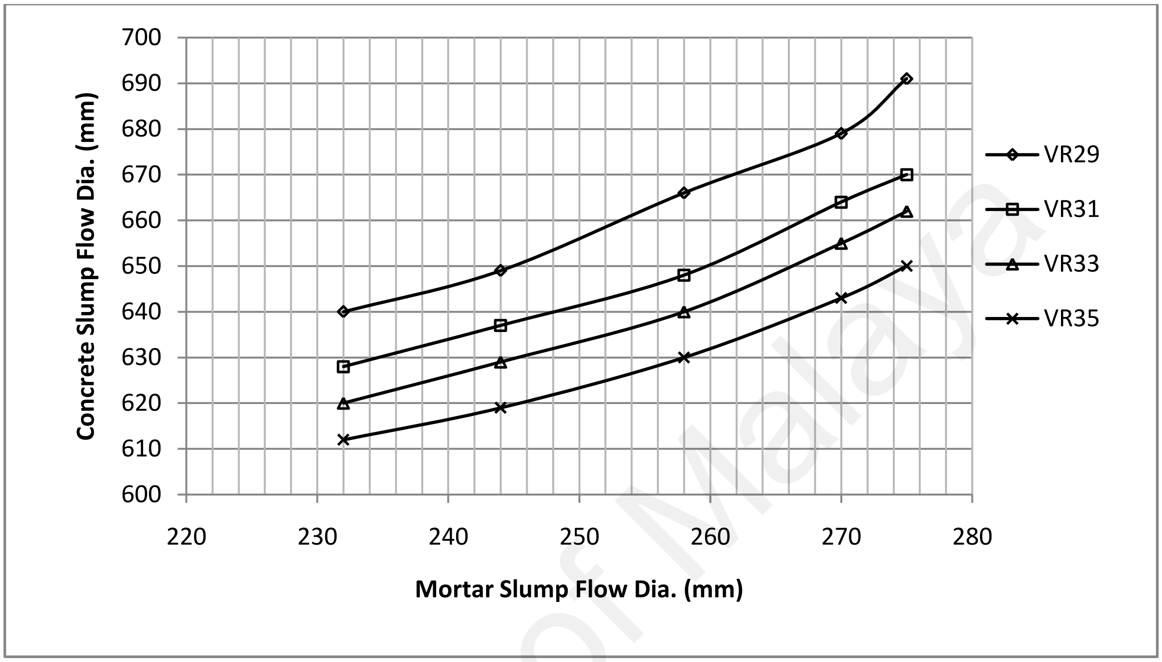


Figure 4.118: The relationship between SFD of mortars and concretes with different aggregate content

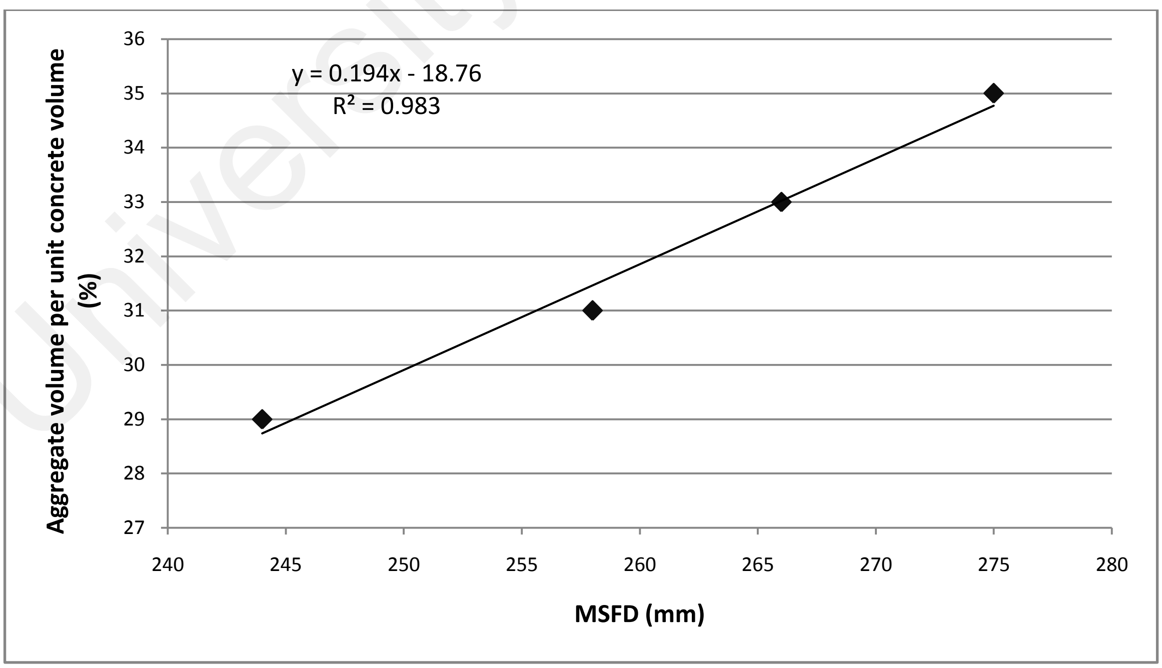


Figure 4.119: Relationship between coarse aggregate volume and MSFD of mortar at SFD of 650 mm for concrete

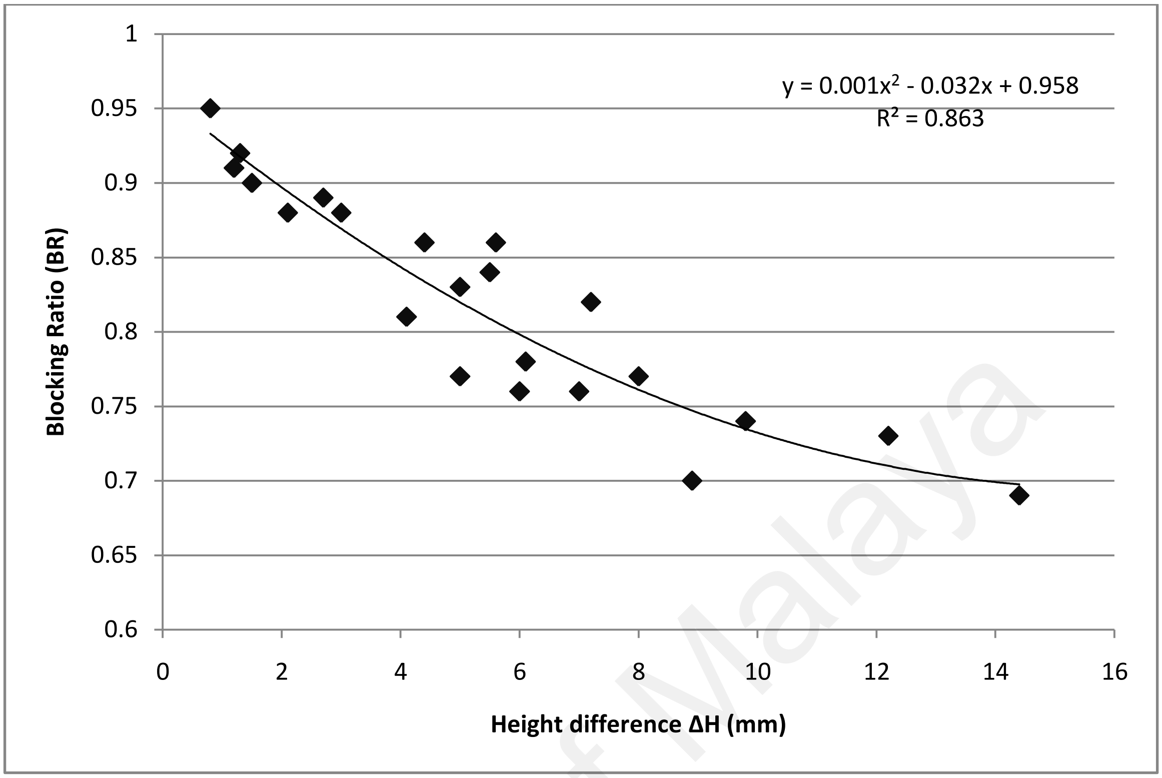


Figure 4.120: The relationship between BR & ΔH

4.8.3 Relationship between slump flow of J-ring and slump cone

The slump flow of slump cone test and slump flow of J-ring test results are correlated with a linear relationship, as shown in Figure 4.121. The correlation coefficient is 0.926. The strong correlation is due to the similar variation of slump flow of slump cone and J-ring tests. The slump flow of slump cone test is higher than the slump flow of J-ring test and this is attributed to the fact that the presence of the steel bar of J-ring leads to a confinement of the coarse aggregate behind it which causes a reduction in slump flow values for J-ring test.

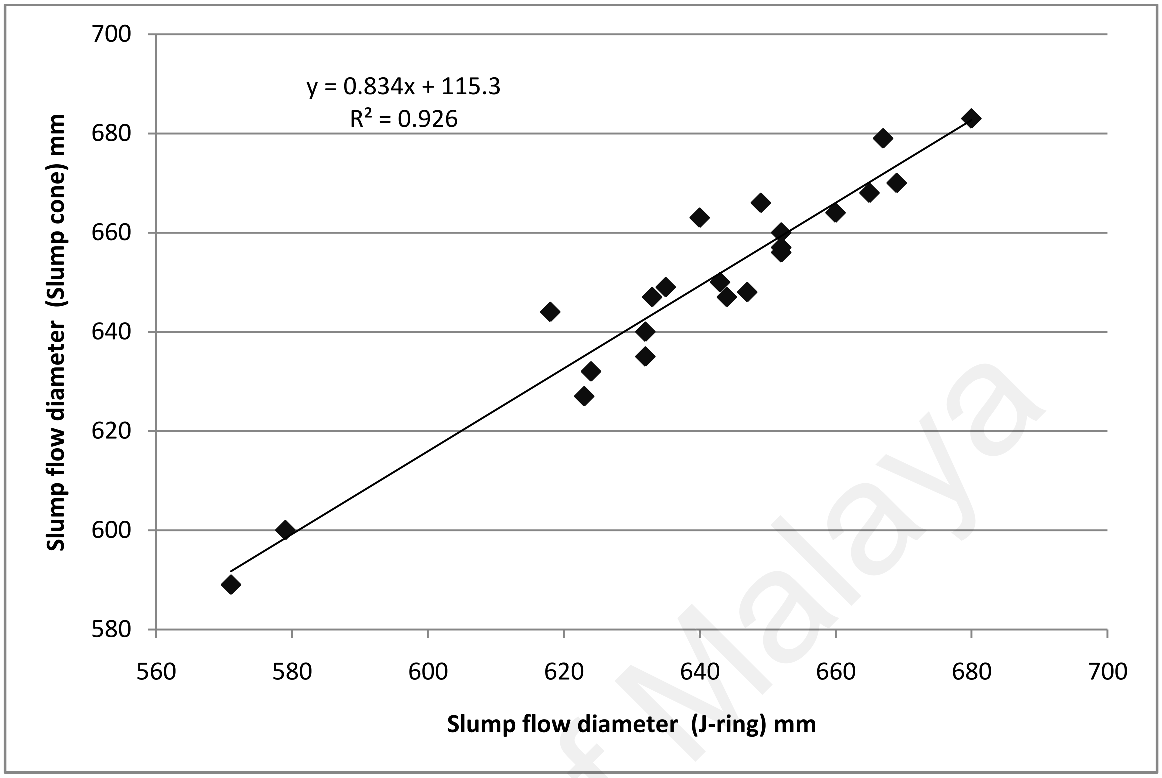


Figure 4.121: The relationship between slump flow diameter of J-ring and slump flow tests

4.8.4 Compressive and splitting tensile strength relationship

The relationship between the experimental results of compressive strength and splitting tensile strength are shown in Figure 4.122. It is clear that the increase in compressive strength is accompanied by an increase in the splitting tensile strength. It appears that the increase in splitting tensile strength of concrete is due to improving the microstructure of concrete, which results from lower porosity and thus achieves better splitting tensile strength.

From this figure, the following equation is derived:-

$\mathbf{f_t=0.358\ f_c^{0.603}}$ **(The present relationship)………………………………… (4.2)**

Where:

f_t: splitting tensile strength
f_c: compressive strength

In comparison with the following relationships stated by ACI modified and other published works which is presented by the equations, the present relationship is deemed to be somewhat close to ACI as shown in Figure 4.122. Table 4.13 shows the comparison of the experimental results with Eqs (4.4 to 4.6) at 28 days. The results show that all equations either overestimated or underestimated the strength by less than 11%. This difference could be due to the circumstances under which the test is conducted which differ from this study. Indeed, different sizes of sample and the use (or lack thereof) of mineral admixture are just a few of the many factors which affect the splitting tensile and compressive strength of concrete.

$f_t=0.53f_{cu}^{0.5}$ ACI 363 Modified (1992).. (4.3)

$f_t=0.22fcu^{0.71}$ Abdul Razak and Wong (2004)……….…………………….…….(4.4)

$f_t= 0.57fcu^{0.5}$ Iravani (1999)……………………..…………………….…….(4.5)

$f_t= 0.32fcu^{0.5}$ Zheng et al. (2001)……………………………….…………..…...(4.6)

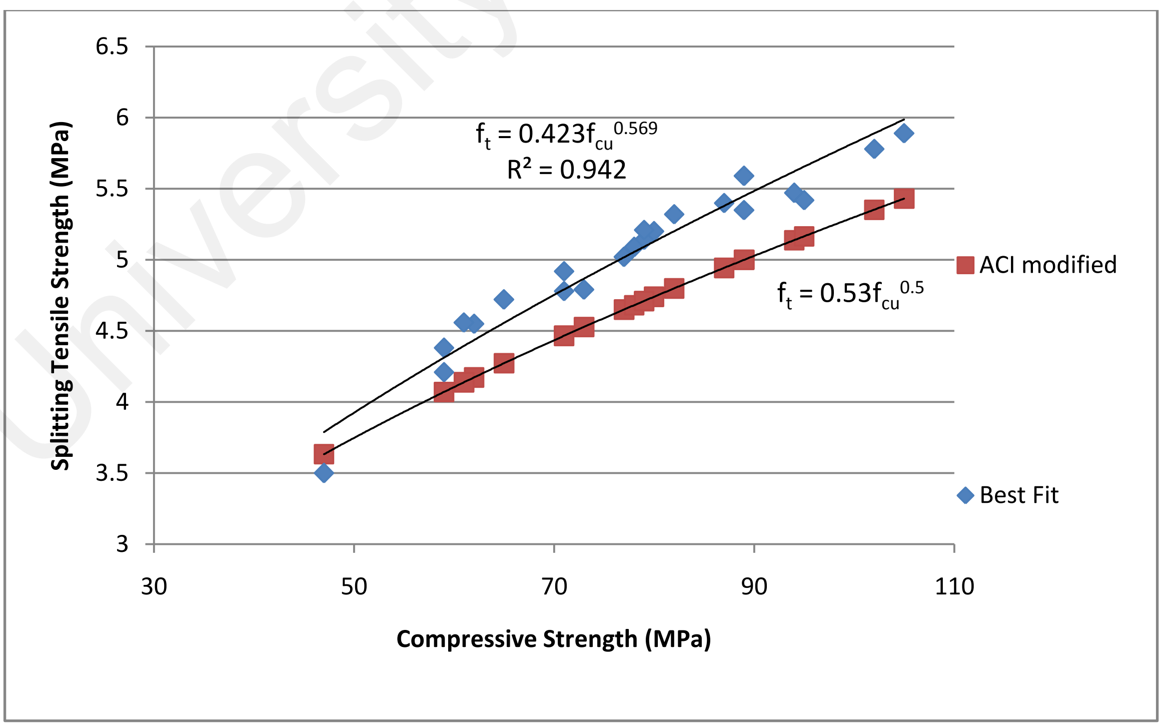


Figure 4.122: The relationship between splitting tensile and compressive strength of HSSCC

Table 4.13: Comparison between experimental and calculated splitting tensile strength at 28 days

Mix No.	f_{cu} (MPa)	Experimental results of f_t (MPa)	Calculated data of f_t (MPa) (difference between Experimental and other Eqs.) (%))				
			Eq 4.2	Eq.4.3	Eq. 4.4	Eq.4.5	Eq.4.6
CC20	62	4.55	4.31(-5)	4.17(-8)	4.12(-9)	4.49(-1)	3.23(-29)
CG20	59	4.38	4.19(-4)	4.10(-6)	3.98(-9)	4.38(0)	3.14(-28)
CC14	65	4.72	4.44(-6)	4.27(-10)	4.26(-10)	4.60(-3)	3.31(-30)
CS14	61	4.56	4.27(-6)	4.14(-9)	4.07(-11)	4.45(-2)	3.20(-30)
CS10	71	4.92	4.68(-6)	4.47(-9)	4.54(8)	4.80(-2)	3.48(-29)
CS10HFR	47	3.5	3.65(4)	3.63(4)	3.39(-3)	3.91(12)	2.76(-21)
FA10C20	77	5.02	4.91(-2)	4.65(-7)	4.81(-4)	5.00(0)	3.64(-27)
FA10G20	73	4.79	4.76(-1)	4.52(-6)	4.63(-3)	4.87(2)	3.54(-26)
FA10C14	79	5.14	4.99(-3)	4.71(-8)	4.89(-5)	5.07(-1)	3.70(-28)
FA10S14	71	4.78	4.68(-2)	4.47(-6)	4.54(-5)	4.80(0)	3.48(-27)
FA10S10	80	5.2	5.03(-3)	4.74(-9)	4.94(-5)	5.10(-2)	3.72(-28)
FA10S10HFR	59	4.21	4.19(-1)	4.1(-3)	3.98(-5)	4.38(4)	3.14(-25)
LP20C20	94	5.47	5.54(1)	5.14(-6)	5.54(1)	5.53(1)	4.07(-26)
LP20G20	89	5.35	5.36(0)	5.00(-7)	5.33(0)	5.38(1)	3.95(-26)
LP20C14	102	5.78	5.82(1)	5.35(-7)	5.87(2)	5.76(0)	4.27(-26)
LP20S14	95	5.42	5.58(3)	5.17(-5)	5.58(3)	5.56(3)	4.10(-24)
LP20S10	105	5.89	5.92(1)	5.43(-8)	5.99(2)	5.84(-1)	4.34(-26)
MK10C20	82	5.32	5.10(-4)	4.80(-10)	5.03(-5)	5.16(-3)	3.77(-29)
MK10G20	78	5.09	4.95(-3)	4.68(-8)	4.85(-5)	5.03(-1)	3.67(-28)
MK10C14	87	5.4	5.29(-2)	4.94(-9)	5.24(-3)	5.32(-1)	3.90(-28)
MK10S14	79	5.21	4.99(-4)	4.71(-10)	4.89(-6)	5.07(-3)	3.70(-29)
MK10S10	89	5.59	5.36(-4)	5.00(-11)	5.33	5.38(-4)	3.95(-29)

Eq 4.2 is the proposed equation in the study.

4.8.5 Flexural with compressive and splitting tensile strength relationship

Figure 4.123 indicates the relationship between the flexural strength and compressive strength. From this figure, it can be seen that the properties of coarse aggregate affect the flexural strength in the same manner as the tensile strength. The coefficient of determination (R^2=0.942) between f_{cu} and f_r for all mixes is illustrated in this figure.

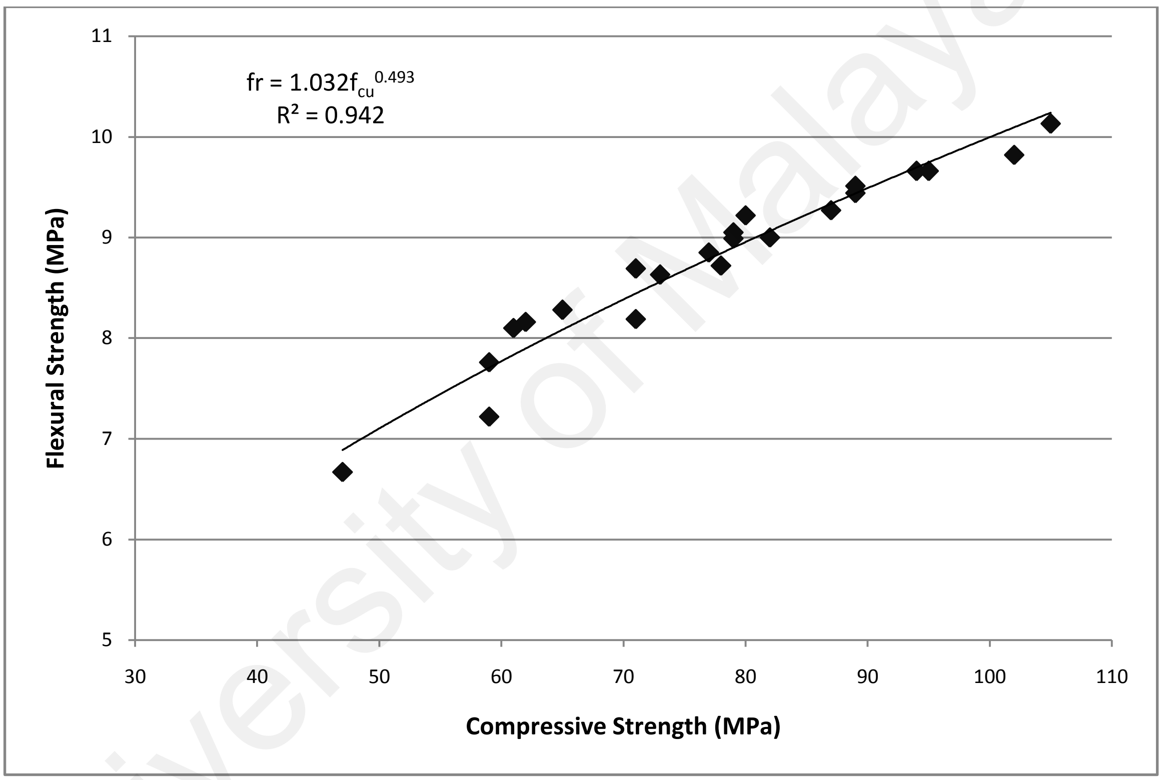


Figure 4.123: The relationship between flexural and compressive strength

Figure 4.124 indicates the comparison of the relationship between the flexural strength and compressive strength between the present study and other studies. The coefficient of determination (R^2=0.942) between f_{cu} and f_r for all mixes is illustrated in the figure.

From this figure, the following equation is derived:-

f_r= 1.032$f_{cu}^{0.493}$ (Present relationship)………………………………………….. (4.7)

Where:

f_r: rupture strength (flexural strength)

f_{cu}: compressive strength of concrete cubes.

In comparison with the following relationship stated by ACI code (ACI 363Modified) 1992 as cited by Abdul Razak and Wong (2005) and as shown in Figure 4.125

$\mathbf{f_r = 0.84f_{cu}^{0.5}}$ **…………………………………………………………………..…… (4.8)**

$\mathbf{f_r=0.949f_{cu}^{0.5}}$ **…………………………………………………………………………(4.9)**

Iravani (1999) propoed equation (4.12) to predict the flexural strength of high strength concrete with compressive strengths between 50-100 MPa. Khan et al. (1996) suggested equation (4.10) to estimate the flexural strength of high strength concrete with compressive strengths between 40-90 MPa

Predicted flexural strength results which were calculated from Eq. 4.9 to 4.13 are listed in Table 4.14. Based on the results in Table 4.14, the experimental results are somewhat deemed overestimate compared to Eq. (4.10 to 4.13) and underestimate compared to Eq. (4.14) at 28 days. Eqns. (4.10 to 4.13) gave the lowest strength difference about 24% underestimated compared to Eq.(4.7), while Eq. (4.14) gave the highest strength difference about 12% overestimated compared to Eq.(4.7). Generally, the present relationship is most appropriate to predict flexural strength of high strength self compacting concrete.

$\mathbf{f_r=1.03f_{cu}^{0.5}}$ **Khan et al. (1996)….……………..…………………...…… (4.10)**

$\mathbf{f_r= 0.84f_{cu}^{0.5}}$ **ACI 363 modified………..…………………………………...(4.11)**

$\mathbf{f_r=0.97f_{cu}^{0.5}}$ **Iravani (1999)……………………………………...……...(4.12)**

$\mathbf{f_r}$**=0.078fcu1.06** **Abdul Razak and Wong (2004)……………………………(4.13)**

$\mathbf{f_r}$**=0.5fcu0.67** **Légeron and Paultre (2000)…………..……………………..(4.14)**

Where, f_{cy} , the cylinder compressive strength and cylinder/cube ratio that is used in this equation is 0.85. The results of this study show that the proposed equation of this study overestimates other equations at 28 days.

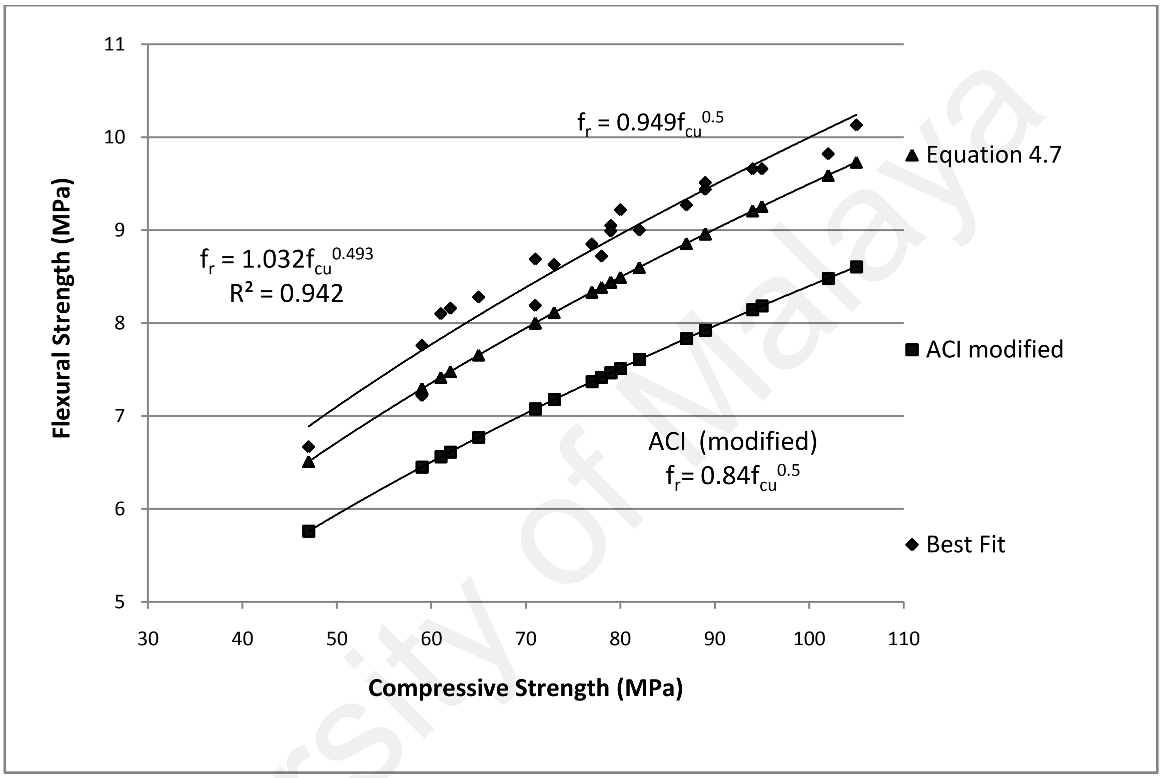


Figure 4.124: The comparison between the present study and other study of the relationship between flexural and compressive strength

Table 4.14: Comparison between experimental and calculated flexural strength at 28 days

Symbols of mix	f_{cu} (MPa)	Experimental results of f_r (MPa)	Calculated data of f_r (MPa) (difference between Experimental and other Eqs.) (%))					
			Eq. 4.7	Eq.4.10	Eq.4.11	Eq. 4.12	Eq. 4.13	Eq. 4.14
CC20	62	8.16	7.89(-3)	6.61(-19)	7.47(-8)	7.63(-6)	6.19(-24)	7.94(-3)
CG20	59	7.76	7.70(-1)	6.45(-17)	7.29(-6)	7.45(-4)	5.87(-24)	7.68(-1)
CC14	65	8.28	8.08(-2)	6.77(-18)	7.65(-8)	7.82(-6)	6.51(-21)	8.19(-1)
CS14	61	8.1	7.83(-3)	6.56(-19)	7.41(-9)	7.57(-7)	6.08(-25)	7.85(-1)
CS10	71	8.19	8.44(3)	7.07(-14)	8.00(-2)	8.17(0)	7.15(-13)	8.69(6)
CS10HFR	47	6.67	6.89(3)	5.76(-14)	6.51(-2)	6.65(0)	4.61(-31)	6.59(-1)
FA10C20	77	8.85	8.78(-1)	7.37(-17)	8.33(-6)	8.51(-4)	7.79(-12)	9.18(4)
FA10G20	73	8.63	8.56(-1)	7.17(-17)	8.11(-6)	8.28(-4)	7.36(-15)	8.85(3)
FA10C14	79	9.05	8.90(-2)	7.46(-18)	8.44(-7)	8.62(-5)	8.00(-12)	9.34(3)
FA10S14	71	8.69	8.44(-3)	7.08(-19)	8.00(-8)	8.17(-6)	7.15(-18)	8.69(0)
FA10S10	80	9.22	8.95(-3)	7.51(-19)	8.49(-8)	8.67(-6)	8.11(-12)	9.4(2)
FA10S10HFR	59	7.22	7.70(7)	6.45(-11)	7.294(1)	7.45(3)	5.87(-19)	7.68(6)
LP20C20	94	9.66	9.69(0)	8.14(-16)	9.20(-5)	9.40(-3)	9.62(0)	10.5(9)
LP20G20	89	9.44	9.43(0)	7.92(-16)	8.95(-5)	9.15(-3)	9.08(-4)	10.1(7)
LP20C14	102	9.82	10.09(3)	8.48(-14)	9.59(-2)	9.79(0)	10.50(7)	11.00(12)
LP20S14	95	9.66	9.74(1)	8.18(-15)	9.25(-4)	9.45(-2)	9.73(1)	10.6(10)
LP20S10	105	10.13	10.24(1)	8.60(-15)	9.73(-4)	9.94(-2)	10.82(7)	11.30(12)
MK10C20	82	9	9.06(1)	7.60(-16)	8.60(-4)	8.78(-2)	8.33(-7)	9.57(6)
MK10G20	78	8.72	8.84(1)	7.41(-15)	8.38(-4)	8.56(-2)	7.90(-9)	9.26(6)
MK10C14	87	9.27	9.33(1)	7.83(-16)	8.85(-5)	9.04(-2)	8.87(-4)	9.96(7)
MK10S14	79	8.99	8.90(-1)	7.46(-17)	8.44(-6)	8.62(-4)	8.00(-11)	9.34(4-)
MK10S10	89	9.51	9.43(-1)	7.92(-16)	8.95(-4)	9.15(-3)	9.08(-8)	10.11(-9)

Eq 4.7 is the proposed equation in the study.

Figure 4.125 shows the relationship plotted for flexural strength against splitting tensile strength for all mixes. The relationship is approximately linear. Although both of these strengths are in tension, but it is indicated that the higher values are of flexural tensile strength compared with splitting tensile strength. This is due to the stress strain curve throughout cross section of the test beam is approximately linear to failure point (Raphael, 1984).

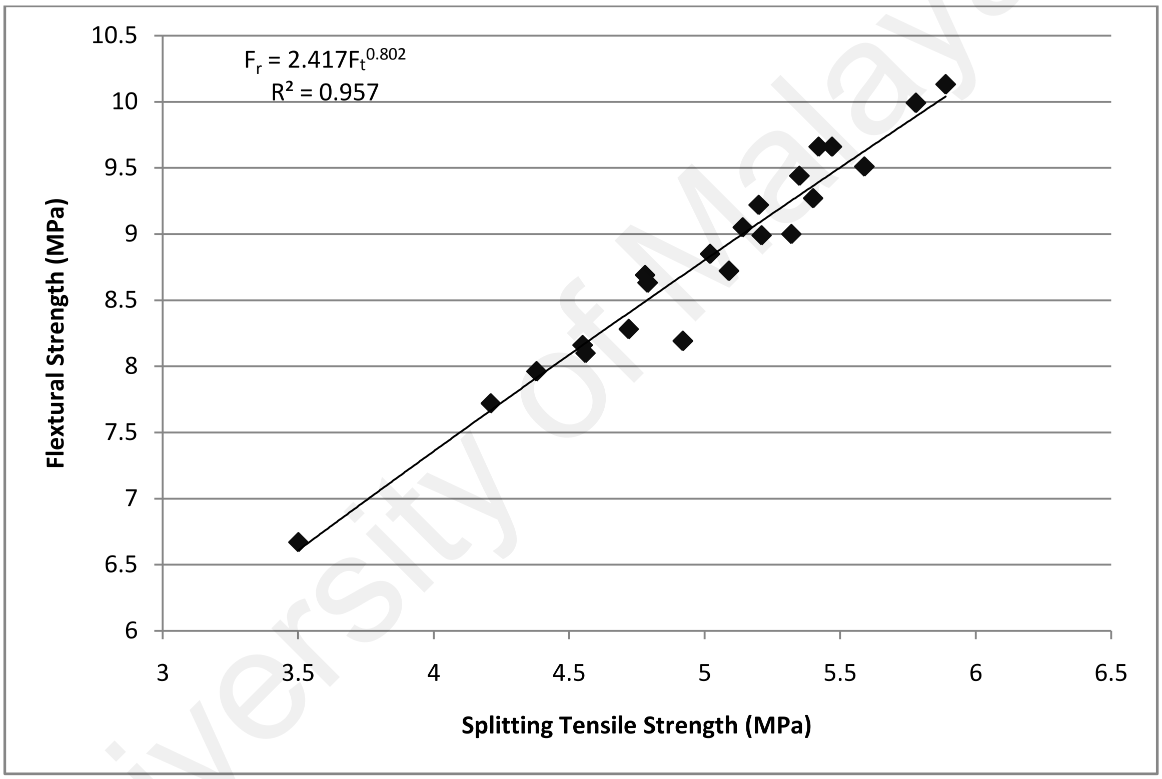


Figure 4.125: The relationship between flexural and splitting tensile strength

4.8.6 Compressive strength and modulus of elasticity relationship

Figure 4.126 shows that the compressive strength of concrete and the elastic modulus of concrete are related, the increase in one is similarly reflected by an increase in the other. This is in agreement with the study performed by Mindess & Young (1981). Figure 4.127

provides a comparison between this relationship and the two other relationships derived from the literature e.g., BS8110 and ACI 318-99.

$E_c = 9.406F_{cu}^{0.313}$ [Present work]…..……………….……………….…………. (4.15)

$E_c = 9.1\ F_{cu}^{0.33}$ [BS8110]….…………………………………………….. (4.16)

$E_c = 4.23\ F_{cu}^{0.0.5}$ [ACI 318-99] …………………………………..…… (4.17)

ACI Committee 363R suggested Eq. 4.17 to predict the modulus of elasticity of normal weight concrete with compressive strength between (21 to 83 MPa) taking into account the difference between cylinder and cube strength. Abdul Razak and Wong suggested Eq. (4.19) to estimate modulus of elasticity of high strength concrete with compressive strength ranged between 75-101 MPa.

$E_c = 3320F_c^{0.5} + 6900$ ……………………………………………………………..… (4.18)

Others equations were proposed and as shown below:

$Ec = 3.32\ fc^{0.5} + 6.9$ Neville (2005)……………………………………….…..... (4.19)

$Ec = 4.14\ fc^{0.5}$ Abdul Razak and Wong (2004)……….…………….…….(4.20)

From the presented results in Table 4.15, the experimental results showed high stiffness by about (7-13%) compared to ACI 363-92 equation and slightly high stiffness by about (1-6%) compared to Neville equation (2005) and almost close to ACI 318-95 and Abdul Razak & Wong (2004) equations.

In comparing a variety of equations available from published studies, it should be careful that the results includes studies with different mixture proportions, with and without supplementary cementitious materials; coarse aggregate contents and types, age of testing and specimen sizes, curing regimes (Abdul Razak and Wong, 2004).

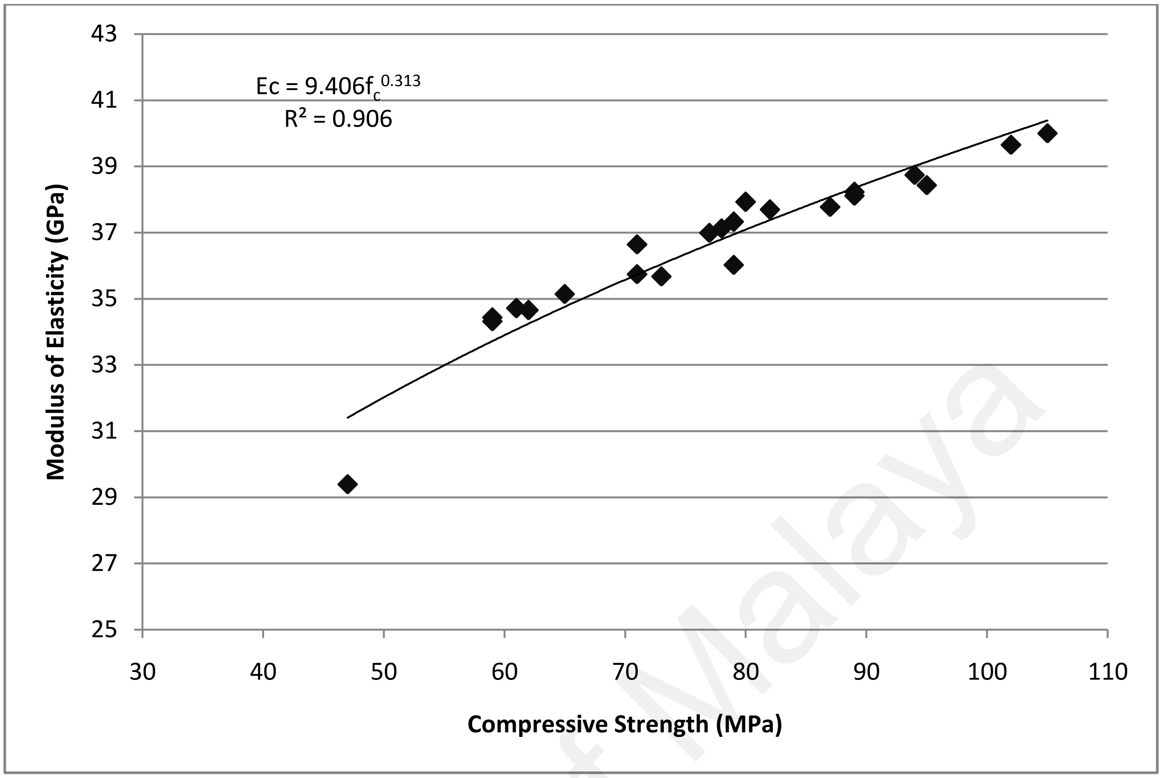


Figure 4.126: Relationship between modulus of elasticity and compressive strength of HSSCC

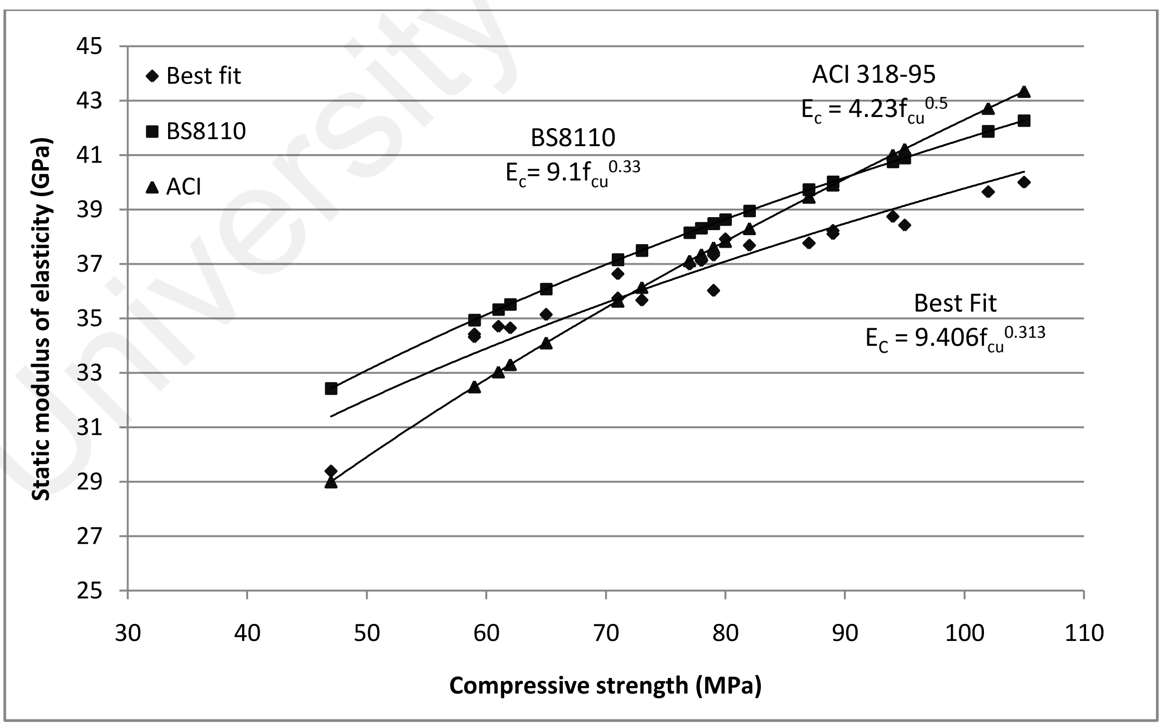


Figure 4.127: The comparison between the present study and other study of the relationship between static modulus of elasticity and compressive strength

Table 4.15: Comparison between experimental and calculated modulus of elasticity at 28 days

Symbols of mix	f_{cu}	Experimental results of Ec	Calculated Data of Ec (GPa) (difference between Experimental and other Eqs.) (%))					
			Eq 4.15	Eq.4.16	Eq.4.17	Eq.4.18	Eq.4.19	Eq.4.20
CC20	62	34.65	34(-1)	35.5(3)	33.3 (-4)	30.3(-13)	33.0 (-5)	32.6(-6)
CG20	59	34.43	34(-2)	34.9(1)	32.5(-6)	29.7(-14)	32.4 (-6)	31.8(-8)
CC14	65	35.14	35(-1)	36.0(3)	34.1(-3)	30.8(-12)	33.7(-4)	33.4(-5)
CS14	61	34.71	34(-2)	35.3(2)	33.3(-5)	30.1(-13)	32.8(-5)	32.3(-7)
CS10	71	35.74	36(0)	37.1(4)	35.6(0)	31.9(-11)	34.9(-2)	34.9(-2)
CS10HFR	47	29.39	31(7)	32.4(10)	29.0(-1)	27.3(-7)	29.7(1)	28.4(-3)
FA10C20	77	36.99	37(-1)	38.1(3)	37.1(0)	33(-11)	36.0(-3)	36.3(-2)
FA10G20	73	35.67	36(1)	37.5(5)	36.1(1)	32.3(-10)	35.3(-1)	35.4(-1)
FA10C14	79	37.32	37(-1)	38.5(3)	37.6(1)	33.3(-11)	36.4(-2)	36.8(-1)
FA10S14	71	36.64	36(-3)	37.1(1)	35.6(-3)	32(-13)	34.9(-5)	34.9(-5)
FA10S10	80	37.92	37(-2)	38.6(2)	37.8(0)	33.5(-12)	36.6(-4)	37.0(-2)
FA10S10HFR	59	34.32	34(-2)	34.4(2)	32.5(-5)	29.7(-13)	32.4(-6)	31.8(-7)
LP20C20	94	38.74	39(1)	40.8(5)	41.0 (6)	35.69(-8)	39.1(1)	40.1(4)
LP20G20	89	38.22	38(0)	40.0 (5)	39.9 (4)	34.91(-9)	38.2(0)	39.1(2)
LP20C14	102	39.65	40(1)	41.9(6)	42.7 (8)	36.89(-7)	40.4(2)	41.8(5)
LP20S14	95	38.43	39(2)	40.9(6)	41.2 (7)	35.84(-7)	39.3(2)	40.4(5)
LP20S10	105	40	40(1)	42.3(6)	43.3(7)	37.33(-7)	40.9(2)	42.4(6)
MK10C20	82	37.69	37(-1)	38.9(3)	38.30(2)	33.8(-10)	37.0(-2)	37.5(-1)
MK10G20	78	37.12	37(-1)	38.3 (3)	37.4(1)	33.1(-11)	36.2(-2)	36.6(-2)
MK10C14	87	37.77	38(1)	39.7 (5)	39.5(4)	34.6(-8)	37.9(0)	38.6(2)
MK10S14	79	36.02	37(3)	38.5(7)	37.6(4)	33.3(-8)	36.4(1)	36.8(2)
MK10S10	89	38.11	38(1)	40.0 (5)	39.9 (5)	34.9(-8)	38.2(0)	39.1(2)

Eq. 4.15 is the proposed equation in the study.

4.8.7 Compressive strength and ultrasonic pulse velocity relationship

A power relationship is seen between the ultrasonic pulse velocity and compressive strength of all mixes as shown in Figure 4.128. It is clear that the ultrasonic pulse velocity increases with the increase of compressive strengths. The coefficient of determination (R^2=0.901) between UPV and f_c for all mixes is illustrated in this figure.

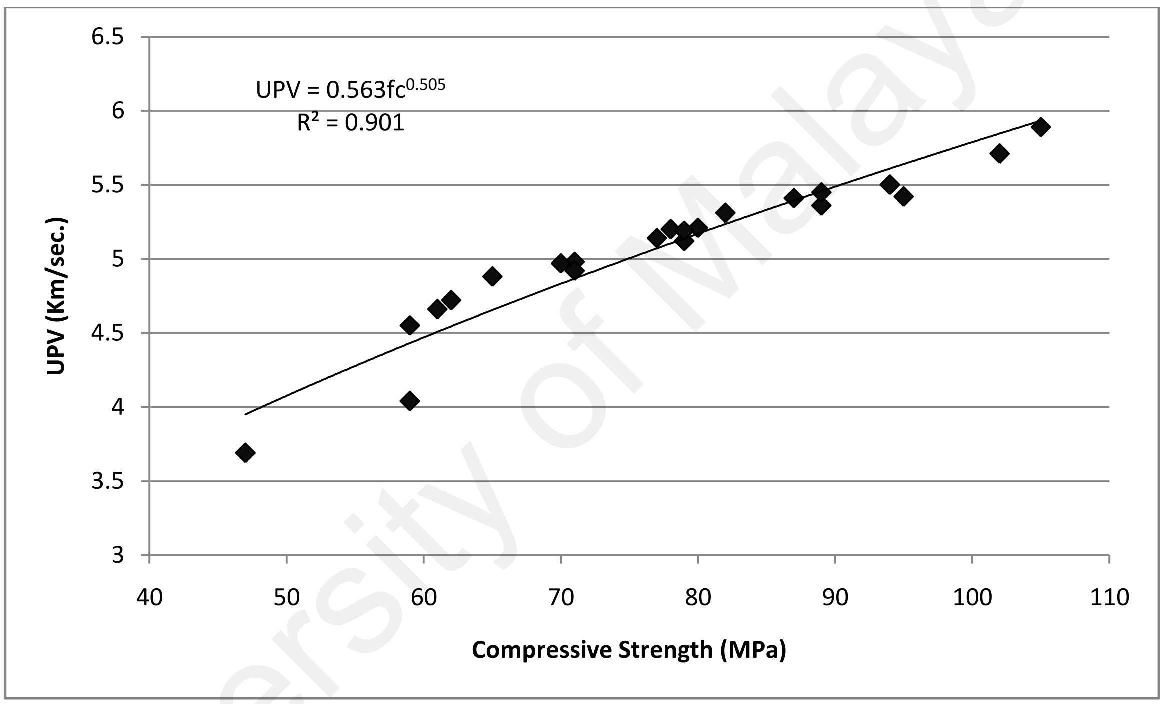


Figure 4.128: The relationship between UPV and compressive strength

CHAPTER 5- CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE RESEARCH

5.1 Conclusions

In this study, dividing the experimental work into three parts e.g paste, mortar, and concrete facilitated the mix design process and thus would save more materials and then would reduce the cost.

At the same water to cement ratio and superplasticizer dosage, the increased sand content produced mortar with reduced flowabilty. Moreover, the increased sand content increased bleeding rate of water in the mortar. Also, the increasing in fineness modulus values of sand decreased the flow spread whilst also increased flow time of cement mortar as well as the water bleeding rate which affects the filling ability of mortar.

The optimum replacement level of FA and MK with cement was 10% from the viewpoint of workability and strength, while the optimum replacement level of limestone with sand was 20%. Self compactibility of mortars was obtained by adding suitable powder materials such as mineral admixtures and superplasticizer. These materials provided a sufficient balance between flowability and viscosity of the mix. The flowability of mixes increased with an increase in the replacement level of FA with cement. Conversely, flowability of the mixes was decreased with an increase in the replacement level of MK with cement. The optimum superplasticiser dosage of FA mortar mixes decreased with the increase in the FA replacement level at the same water cement ratio, while it increased with an increase in the metakaolin replacement level for MK mortars. Moreover, the use of fillers, namely LP and K provided good stability for the mixes. Generally, after the optimum dosage of superplasticizer, the increment in the flow spread was found to be insignificant. In most cases when the superplasticiser dosage was more than optimum dosage, bleeding occurred. Generally, the shape and fineness of powder had a great effect on superplasticiser dosage. The superplasticiser dosage and cement content played a vital role in acceleration and retardation of the setting time of the mortar mix, while adding LP accelerated the setting time of mixes. The strength increased with an increase in the replacement level of LP, while the strength decreased with an increase in the replacement level of FA and MK beyond the 10% replacement level. The water absorption decreased with an increase in the replacement level of LP. With 10% replacement level for both fly ash and metakaolin there was reduction in the water absorption of mortar mixes when compared with the control mix.

FA mixes had high flowability, and had a better passing ability when compared to other mixes. Generally, filler and pozzolanic materials gave better static stability of the fresh mixes than cement. The use of limestone powder in the mixes enhanced compressive, splitting tensile and flexural strengths and modulus of elasticity of the mixes better than the pozzolan and cement. Metakaolin concretes had compressive, splitting tensile and flexural strengths and modulus of elasticity higher than fly ash concretes. From the water absorption test results, the limestone mixes showed lower water absorption values when compared with other mixes. Moreover, metakaolin reduced water absorption of the mixes with level more than fly ash. The drying shrinkage was mainly affected by the type of powder used in the mixes. The highest drying shrinkage was for limestone mixes followed by fly ash mixes and then metakaolin mixes. Control mixes exhibited less drying shrinkage compared to other mixes. The use of powders gave good workability and strength and thus would reduce of the use chemical admixtures and cement and consequently reduce the cost. It is feasible to produce HSSCC with different powders.

Metakaolin (MK) was characterized as a pozzolanic material because it has a pozzolanic activity; while kaolin (K) was characterized as filler because it is an inert powder, although both of MK and K have approximately the same silica content and fineness. The effect of MK and K on workability was approximately the same, while their affect strength was a different. MK gave a great affect strength comparing to K due to the pozzolanic activity of MK which is not available in K. the pozzolanic activity which is available in MK due to the changing occurred in structure of MK due to burning comparing to K.

For the same mix proportion of mortar, the increment in coarse aggregate volume decreased flowability, passing ability and segregation resistance of the fresh concretes. Moreover, larger maximum size of coarse aggregate increased the flow spread of mixes but decreased filling ability. Smaller maximum size of coarse aggregate increased passing ability through steel bars and stability of the fresh concretes and well grade aggregate improved flowability of the fresh mixes. Through the passing of fresh concrete between the steel bars, the gap graded and single sized aggregate was more prone to blocking behind bars than well grade aggregate. In addition the gap graded and single size aggregate were more prone to segregation in the mixes compared to continuous aggregate. However, the aggregate with high flakiness ratio reduced slump flow diameter of mixes and the ease in discharging of fresh concrete through the V-funnel opening. The presence of aggregate with a high flakiness ratio in the mixes reduced the concrete passing through the steel bar and lowered the stability of the mixes. The flowability of concrete increased with an increase in the flowability of mortar at the same coarse aggregate volume. Generally, the different coarse aggregate properties had a great role in affect the fresh properties of HSSCCs. The flowability of mortar has a great role in affecting the flowability of concrete.

The results of compressive, tensile and flexural strength for all mixes varied between 47 to 105 MPa, 3.5 to 5.9 MPa and 6.67 to 10.13 MPa respectively at 28 days depending on the variables of this study. The modulus of elasticity of all powder materials varied between 35.5 to 40 GPa according to the type of the pozzolan and filler used. The modulus of elasticity of control mixes ranged between 34 to 35 GPa. The range of the water absorption results varied between 2.22 to 6.32% for all of the mixes at 180 days. The drying shrinkage of all mixes varied between 305 to 470 µs at 360 days and according to the type of the powder used and characteristics of the coarse aggregate. The ultrasonic pulse velocity for the mixes varied between 3.69 to 5.89 km/sec and was differently affected according to the type of powder used as well as the properties of the coarse aggregate used i.e. maximum size, grading and shape.

The 33% coarse aggregate volume exhibited higher compressive, tensile, flexural strength as well as modulus of elasticity compared to other aggregate volumes in the mixes. The increase in coarse aggregate volume reduced the drying shrinkage as it acted to restrain the drying shrinkage which was occurring in the cement paste.

The mechanical properties with respect to compressive, tensile strengths as well as modulus of elasticity of HSSCC were significantly affected by the different coarse aggregate properties. Where, they were enhanced by using small maximum size, graded, and low flakiness ratios aggregate compared to other properties of aggregate. Drying shrinkage of HSCC was also reduced by utilizing large size, graded as well as low flakiness ratios aggregate compared to other aggregate properties. It could be said that it is feasible to produce HSSCC with different aggregate properties.

An equation was proposed to estimate the coarse aggregate volume by flowability of mortar. From the different relationship of tests results, blocking ratio and height difference of J-ring test had a good correlation between them. Slump flow of slump cone and slump flow of J-ring test had a good correlation with a coefficient of 0.93. A good relationship was found between the flowability of mortar and concrete with a coefficient of 0.98. A good relationship was found between the fresh tests of SCC with a coefficient ranged 0.87 to 0.93. There were a good relationship was found between compressive strength and splitting tensile strength and it was an overestimate comparing to other correlation such as international standard and published works. Moreover, the relationship between flexural and compressive strength were established was good with a coefficient of 0.94. It was also compared to other relationship international standard e.g ACI and BS and it was overestimate with some of them and underestimates with others. In addition, it was comparable relationship compared to four published studied. The summary of this paragraph is good correlations were found between the results of the fresh and hardened tests for HSSCC, and an equation is proposed to estimate the coarse aggregate volume for HSSCC using the flowability of mortar.

5.2 Recommendations for future research

There are many aspects that were not studied in this research. There are many factors that affect production high strength self compacting concrete. More researches are required to produce high strength self compacting concrete. The effect of particle size distribution of metakaolin, fly ash and limestone on producing high strength self compacting concrete should be studied. More researches are required regarding the effect of different properties of fine aggregate on the engineering properties of HSSCC i.e. type, particle size distribution, and shape of sand. However, more studies should be investigated about the study of the behaviour of HSSCC under the effect of the texture and type of coarse aggregate. Selecting the proper grade of fine and coarse aggregate should be investigated in producing ultra high strength self compacting concrete. There are many studies should be investigated about the developing of the test methods of fresh HSSCC. The study of the comparison between the viscosity and yield value of self compacting mortar and concrete should be also investigated. Moreover, more studies are required for producing ultra high strength self compacting concrete by using local materials.

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