MECHANICAL PROPERTIES AND STRUCTURAL PERFORMANCE OF SUSTAINABLE LIGHTWEIGHT AGGREGATE CONCRETE USING BLENDED OIL PALM BIO-PRODUCTS

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2017

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

FACULTY OF ENGINEERING
UNIVERSITY OF MALAYA
KUALA LUMPUR

2017

UNIVERSITY OF MALAYA

ORIGINAL LITERARY WORK DECLARATION

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Registration/Matric No: KHA120129

Name of Degree: Doctor of Philosophy

Title of Thesis:

Mechanical Properties and Structural Performance of Sustainable Lightweight Aggregate Concrete using Blended Oil Palm Bio-products

Field of Study: Structural Engineering & Materials

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ABSTRACT

A number of materials have been proposed for producing structural grade of lightweight concretes. However, the main concerns of civil engineers and researchers to develop a proper structural lightweight concrete are cost effective, being environmental friendly and durable, as well as behaves satisfactorily under structural applications. Although several types of artificial lightweight aggregates are available to produce structural lightweight concrete, however, these types of aggregates require huge amount of energy in their preparation. Therefore, use of industrial waste as an alternative lightweight aggregate to build environmentally sustainable structures has several practical and economic advantages. Oil palm shell (OPS) is a solid waste material from the palm oil industry that has been successfully used to produce high strength lightweight concrete. However, this type of concrete is very sensitive to a poor curing environment. In addition, OPS concrete with normal and high strengths have relatively high drying shrinkage at early and later ages. Therefore, to produce a cleaner and greener concrete, this study tried to resolve disadvantages of OPS lightweight concrete using another type of lightweight aggregate waste origin from palm oil industry namely oil-palm-boiler clinker (OPBC). Therefore, a new high strength lightweight concrete containing blended coarse lightweight aggregates was developed and its mechanical and time-dependent properties as well as structural performance in term of shear behaviour of simply supported reinforced concrete beams were investigated. It was found from this study that use of blended OPS and OPBC can produce a workable high strength lightweight concrete with a 28-day compressive strength in the range of 40 to 53 MPa and dry density less than 2000 kg/m^3. The individual use of OPBC as coarse aggregate can produce high strength semi-lightweight concrete with 28-day compressive strength of 60 MPa and dry density of 2050 kg/m^3. Test results showed that when OPS concrete is very sensitive to poor curing and minimum period of 7 days moist curing is recommended for this concrete, the sensitivity of compressive strength of concrete to the lack of curing is significantly reduced when OPS was partially replaced with OPBC. The substitution of OPS with OPBC significantly improved the splitting tensile and flexural strengths as well as the modulus of elasticity of the concretes. Concretes containing blended coarse lightweight aggregates of OPS and OPBC showed initial and final water absorption of less than 3% and 10%, respectively, and can be considered as good quality concrete. Drying shrinkage strain value of OPS and OPS-OPBC blended coarse concretes is found to be similar at early ages. However, the long-term shrinkage of blended coarse concretes was significantly lower than OPS concrete. Under 7-day initial moist curing condition, OPS concrete showed significantly higher drying shrinkage of about 47%, 41% and 39% compared to the shrinkages of normal weight, artificial lightweight and OPBC concretes at one year age. In structural performance, OPS reinforced concrete beams without shear links showed lower ultimate load carrying capacity compared to the blended coarse lightweight concrete beams. However, higher substitution of OPBC in OPS concrete with high grade concrete significantly improved the structural performance and failure nature of the beams.

ABSTRAK

Beberapa bahan telah dicadangkan untuk menghasilkan konkrit ringan. Walau bagaimanapun, kebimbangan utama jurutera awam dan penyelidik adalah untuk membangunkan konkrit struktur ringan yang betul dimana haruslah kos efektif, mesra alam, tahan lama, dan secara khususya bersifat memuaskan di bawah struktur aplikasi. Walaupun, beberapa jenis agregat buatan ringan boleh didapati tetapi ia memerlukan sejumlah tenaga yang besar untuk penyediaan dan menggunakan sejumlah wang yang besar. Oleh itu, penggunaan sisa industri sebagai agregat alternatif untuk membina struktur alam sekitar yang mampan mempunyai beberapa kelebihan praktikal dan ekonomi. Tempurung kelapa sawit (OPS) adalah bahan buangan pejal daripada industri minyak sawit yang telah berjaya digunakan untuk menghasilkan konkrit ringan tahan lama berkuatan tinggi. Walau bagaimanapun, jenis konkrit ini adalah sangat sensitif kepada pengawetan alam sekitar yang kurang. Tanbahan pula, konkrit tempurung kelapa sawit yang normal berkuatan tinggi mempunyai pengecutan kering yang agak tinggi pada peringkat awal dan seterusnya. Oleh itu, untuk menghasilkan konkrit yang lebih bersih dan hijau, kajian ini menggunakan dua bahan buangan daripada industri minyak sawit sebagai agregat kasar; untuk membangunkan struktur konkrit ringan berkuatan tinggi serta sifat-sifat mekanikal dan bergantung kepada masa, dan sifat struktur telah disiasat. Kajian mendapati bahawa penggunaan bahan-bahan campuran seperti tempurung kelapa sawit dan minyak sawit dandang klinker boleh menghasilkan konkrit ringan berkuatan tinggi kebolehkerjaan tinggi dan kekuatan mampatan dalam 28 hari kira-kira 40 hingga 53 MPa dan ketumpatan kering kurang daripada 2000 kg/m^3. Penggunaan individu minyak sawit dandang klinker sebagai agregat boleh menghasilkan kekuatan konkrit semi-ringan yang tinggi dengan 28 hari kekuatan mampatan 57 MPa dan ketumpatan kering 2050 kg/m^3. Walau bagaimanapun, konkrit tempurung kelapa sawit adalah sangat sensitif kepada pengawetan yang kurang dan tempoh minimum 7 hari pengawetan lembap

adalah disyorkan. Kepekaan kekuatan mampatan konkrit tempurung kelapa sawit kekurangan pengawetan semakin berkurangan disebabkan oleh penggunaan agregat sebati. Semua konkrit sebati masing-masing menunjukkan penyerapan air awal dan akhir kurang daripada 3% dan 10%, boleh dianggap konkrit yang baik. Penggantian tempurung kelapa sawit dengan minyak sawit-dandang klinker dengan ketara meningkatkan tegangan belahan dan kekuatan lenturan dan modulus keanjalan konkrit. Lenturan kepada nisbah kekuatan mampatan dan modulus keanjalan konkrit sebati berada dalam julat berat normal dan konkrit agregat ringan berkekuatan tinggi. Pengecutan pengeringan pal minyak kelapa dan konkrit sebati didapati sama pada peringkat awal. Walau bagaimanapun, pengecutan jangka panjang konkrit sebati adalah jauh lebih rendah berbanding konkrit tempurung kelapa sawit. Dengan cara lain, sumbangan minyak sawit dandang klinker dalam konkrit tempurung kelapa sawit dengan pengurangan kecil air kepada nisbah simen mengurangkan pengecutan kering kira-kira 40% berbanding dengan konkrit tempurung kelapa sawit kawalan. Di bawah 7 hari pengawetan lembap, konkrit tempurung kelapa sawit menunjukkan pengecutan pengeringan jauh lebih tinggi kira-kira 47%, 41% dan 39% berbanding dengan kesusutan berat normal, buatan ringan dan konkrit minyak sawit dandang klinker. Dari segi prestasi struktur, tulang rasuk konkrit tempurung kelapa sawit menunjukkan kapasiti beban yang lebih rendah daripada rasuk konkrit ringan sebati. Walau bagaimanapun, penggantian yang lebih tinggi bagi minyak sawit-dandang klinker dalam konkrit tempurung kelapa sawit dengan konkrit gred tinggi dengan ketara meningkatkan prestasi struktur dan sifat kegagalan rasuk.

ACKNOWLEDGMENTS

I wish to express my deepest gratitude to my supervisors *Dr Payam Shafigh* and *Professor Ir Dr Mohd Zamin Jumaat* for their time, advice, encouragement, and expertise during the research. This thesis would not have been possible without their elaborate comments, guidance and full encouragement at various stages.

I would also like to thank my best friend *Engr Waqar Ahmad Unar* and the laboratory assistant *Mr. Sreedharan* for their cooperation in providing and preparing the required laboratory equipment.

Special thank is extended to the financial support of University of Malaya, High Impact Research Grant (HIRG) No. UM.C/625/1/HIR/MOHE/ENG/36 (16001-00-D000036), “Strengthening Structural Elements for Load and Fatigue”.

Every challenging work needs self-efforts as well as guidance of elders especially those who were very close to our heart. My humble effort I dedicate to sweet and loving;

FATHER & MOTHER

Whose affection, love, encouragement, and prays of day and nights make me able to get such success and honor.

MY WIFE & SON (ARSU), UNCLE, BROTHERS & SISTERS

Who supported me each step of the way, and for their extreme love,

Along with all hardworking and Respected

TEACHERS

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3R	:	Reduce, Reuse, Recycle
2T2D	:	Two times 2 days curing
2T6D	:	Two times 6 days curing
2D	:	Two days curing in water
4D	:	Four days curing in water
6D	:	Six days curing in water
7W	:	Seven days moist curing
AASHTO	:	American Association of State Highway and Transportation Officials
AC	:	Air curing
ACI	:	American Concrete Institute
AS	:	Australian Standards
ASTM	:	American Society for Testing and Materials
BS	:	British Standards
C	:	OPBC semi-lightweight concrete
CSA	:	Canadian Standard Association
CV	:	Coefficient of variation
DoE	:	Department of Environment
E	:	Young's modulus of elasticity
E-value	:	Value of Young's modulus of elasticity
EFB	:	Empty fruit bunches
EP	:	Error percentage
EQ	:	Ecosystem quality
FA	:	Fly ash
FFB	:	Fresh fruit bunches

FIP	:	Federation Internationale de la Precontrainte
FW	:	Continuous moist curing
GGBFS	:	Ground granulated blast furnace slag
HSC	:	High Strength Concrete
HSLWAC	:	High strength lightweight aggregate concrete
IS	:	Indian Standards
ISO	:	International Standard Organization
IUTM	:	Instron Universal Testing Machine
JSCE	:	Japan Society of Civil Engineers
K	:	Constant parameter
L	:	Leca lightweight aggregate concrete
LCA	:	Life cycle assessment
LCI	:	Life cycle inventory
LECA	:	Lightweight expanded clay aggregate
LEED	:	Leadership in Energy and Environmental Design
LUC	:	Land-use change
LVDT	:	Linear voltage differential transducers
LWA	:	Lightweight Aggregate
LWAC	:	Lightweight aggregate concrete
LWC	:	Lightweight concrete
LWAFC	:	Lightweight aggregate foamed concrete
MC	:	Model Code
MOR	:	Modulus of rupture
MS	:	Malaysian Standards
N	:	Normal weight concrete mix
NC-SF	:	Normal weight concrete containing silica fume

NWA	:	Normal weight aggregate
NWC	:	Normal weight concrete
OPBC	:	Oil-palm-boiler clinker
OPBCC	:	Oil-palm-boiler clinker concrete
OPBC-C	:	Oil-palm-boiler clinker as coarse aggregate
OPBC-F	:	Oil-palm-boiler clinker as fine aggregate
OPC	:	Ordinary Portland cement
OPS	:	Oil palm shell
OPS-FA	:	Oil palm shell concrete containing fly ash
OPSFC	:	Oil palm shell foamed concrete
OPS-SF	:	Oil palm shell concrete containing silica fume
P	:	Oil palm shell concrete mix
PKS	:	Palm Kernel shell
PKSC	:	Palm Kernel shell concrete
PS	:	Plastic sheet
POC	:	Palm oil clinker
POFA	:	Palm oil fly ash
POME	:	Palm oil mill effluent
RC	:	Reinforced concrete
RSM	:	Response Surface Methodology
SCC	:	Self-compacting concrete
SLWAC	:	Structural lightweight aggregate concrete
SP	:	Super-plasticizer
SRC	:	Sulphate Resistant Cement
UNEP	:	United Nations Environment Programme
UN	:	United Nations

USGBC	:	United States Green Building Council's
f_{cu}	:	Cubical compressive strength
f_{cy}	:	Cylinder compressive strength
f_t	:	Splitting tensile strength
f_r	:	Flexural strength
w/c	:	Water cement ratio
MPa	:	Mega Pascal
ω	:	Air dry density of concrete (kg/m^3)
b_w	:	Width of the beam
a/d	:	Shear span to effective depth ratio
M_{ult}	:	Ultimate moment
ρ	:	Tension reinforcement ratio
V_c	:	Shear resistance
β	:	Shear resistance factor of concrete
d	:	Effective depth of the beam
λ	:	Factor for low density concrete
w	:	Dry density (kg/m^3)
S_t	:	Drying shrinkage strain
S_∞	:	Ultimate shrinkage strain
Y_{sh}	:	Product of several factors
v/s	:	Volume to surface area ratio
Y_{tc}	:	Curing time coefficient
Y_{RH}	:	Relative humidity coefficient
Y_{vs}	:	Factor related to volume to surface area ratio
Y_s	:	Slump related factor (slump in mm)
Y_ψ	:	Fine aggregate to total aggregate ratio

Y_c	:	Cement related factor (cement content in kg/m^3)
Y_α	:	Air content factor (%)
K_1'	:	Curing time coefficient
K_2'	:	Relative humidity coefficient
K_3'	:	Factor related to volume to surface area ratio
K_4'	:	Slump related factor (slump in mm)
K_5'	:	Fine aggregate to total aggregate ratio
K_6'	:	Cement related factor (cement content in kg/m^3)
K_7'	:	Air content factor (%)
ε_0	:	Strain at peak stress
ε_{cs}	:	Total drying shrinkage strain
ε_{cd}	:	Drying shrinkage strain
ε_{ca}	:	Autogenous shrinkage
$\varepsilon_{sh}(t\text{-}t_0)$	:	Drying shrinkage strain
ε_{shu}	:	Ultimate shrinkage strain
$\varepsilon_{sh\infty}$	:	Ultimate shrinkage strain
ε_{so}	:	Notional shrinkage coefficient
β_{sc}	:	Cement type factor
A_c	:	Cross-sectional area in (mm^2)
μ	:	Exposed perimeter (mm)
k_{vs}	:	Size factor
K_H'	:	Ambient relative humidity factor
k_{t0}'	:	Factor for maturity of concrete
h	:	Relative humidity in decimals
α	:	Factor for type of cement
τ_{sh}	:	Size and shape dependent factor

α_1	:	Cement type factor
α	:	Angle of deflection lines in structural behaviour of beams
β	:	Angle of deflection lines in structural behaviour of beams
ρ_1	:	Longitudinal reinforcement ratio
d_o	:	Effective depth related factor
α_2	:	Curing type factor
k_h	:	Humidity dependent factor
$V_{\mathrm{Rd,c}}$	:	Shear resistance of concrete
$C_{Rd,c}$	:	Additional factor for shear capacity
η_1	:	Additional factor for shear capacity
γ_c	:	Partial safety factor
A_s	:	Area of steel reinforcement
W/mK	:	Unit of thermal conductivity (Watts per meter-Kelvin)

CHAPTER 1: INTRODUCTION

1.1 Introduction

Concrete and steel are the most commonly used materials in construction industry. Among that the structural concrete plays very important role in all types of civil engineering structures. Because it has an excellent resistance to water and can be formed into a variety of shapes and sizes. The principal constituents to make concrete are cement, fine and coarse aggregates and water, and these materials are commonly available in all over the world.

Generally, the concrete can be classified into three categories based on its unit weight or density, namely normal weight, lightweight and heavy weight concrete. Among, such types, the lightweight concrete (LWC) has attracted a considerable attention due to its higher strength to weight ratio compared to the normal weight concrete (NWC). In addition, the high demand of NWC made with normal weight aggregates has extremely reduced the natural stone deposits and caused an irreparable damage to the environment. As a result, the emphasis on the sustainable materials has been increased recently, which motivated the researchers to focus their investigation on the use of recycled or waste materials into the potential construction materials.

There are several methods to produce lightweight concrete. The most popular way of achieving LWC production is by using lightweight aggregate (LWA). The lightweight concrete is not a new invention in concrete technology; it has been used since ancient times and the most interesting field for the researchers because of its advantages such as savings in scaffolding, formwork and reinforcement, it also reduces the foundation, erection and transport costs. Furthermore, the other advantages of LWC are better heat insulation, fire resistance, frost resistance, sound absorption, superior anti-condensation properties and also increased damping (Shafigh, Mahmud, & Jumaat, 2010).

1.2 Lightweight Concrete

The concrete can be considered as a lightweight concrete with the density lower than the usual range of concretes made with the normal weight aggregates. Lightweight concrete was further classified into three categories based on their production methods (Neville & Brooks, 2008);

i. The concrete made from the porous aggregate of lower specific gravity can be considered as the lightweight aggregate concrete.
ii. The concrete made by introducing the air voids in the cement paste can be considered as cellular, aerate, foamed or gas lightweight concrete.
iii. The concrete made by omitting fine aggregates can be considered as no-fine lightweight concrete.

Based on the application of concrete, the lightweight concrete can also be classified in three categories (Neville, 2008);

i. The concrete with the minimum 28-day cylinder compressive strength of 17 MPa and the density in the range of 1350 to 1900 kg/m^3 can be considered as structural lightweight concrete because it can be used for structural purposes (ASTMC330, 2003).
ii. The concrete with the cylinder compressive strength in the range of 7 to 17 MPa with the dry density range between 300 to 800 kg/m^3 can be considered as lightweight concrete used in masonry units (ASTMC331, 2003).
iii. The concrete with the standard cylinder compressive strength between 0.7 and 7 MPa with the dry density range between 300 and 800 kg/m^3 can be considered as low density or insulation concrete (ASTMC332, 2003).

Generally, structural lightweight concrete is similar to the normal weight concrete except it has lower density. It was found that the lightweight concrete can be produced with the 28-day cube compressive strengths from 1 to over 60 MPa and the oven-dry density range of approximately 300 to a maximum of 2000 kg/m^3, and the thermal conductivities of 0.2 to 0.1 W/mk (Newman & Owens, 2003). Typically, the density of structural lightweight concrete range between 1400 to 2000 kg/m^3 (Shannag, 2011) and the 28-day compressive strength ranges between 20 to 35 MPa (Kosmatka, Kerkhoff, & Panarese, 2002). It is also possible to produce high strength structural lightweight concrete with the compressive strength range of 35 to over 70 MPa by introducing various pozzolans such as silica fume, fly ash and metakaolin etc. (Holm & Bremner, 2000).

1.3 Lightweight Aggregates

The lightweight aggregates (LWAs) are generally classified in two groups, natural and artificial (manufactured). The main natural LWAs are pumice, diatomite, volcanic cinders, scoria and tuff (Neville, 2008), and the artificial aggregates are further classified in two groups. The naturally arising materials that need further processing (produced by the application of high temperature) such as shale, expanded clay, slate, vermiculite and perlite, and the materials that occur as industrial by products such as sintered slate, sintered pulverized fuel ash (fly ash) and colliery waste, expanded or foamed blast-furnace slag (Shafigh et al., 2010).

Since 2nd A.D. different types of LWA such as foamed slag, and expanded clay has been used as construction material. Oil Palm Shells (OPS), sometimes called Palm Kernel Shells (PKS) and Oil-palm-boiler clinkers (OPBC) are an alternative LWA in tropical regimes and countries that have a palm oil industry. Malaysia is one of the top listed countries which has huge assets of OPS, and contributes about 58% of the total supply of palm oil in the world (Ahmad, Ibrahim, & Tahir, 2010). OPS and OPBC are produced in large quantities by the oil mills. For instance, in Malaysia and Nigeria it was estimated that over 4 million tonnes of OPS solid waste is produced annually and only a fraction is used for the production of the fuel and activated carbon. In Malaysia, the use of OPS as a LWA or porous aggregate in producing LWC was introduced by Sallam & Abdullah (1985).

1.4 Problem Statement

The use of high strength normal weight concrete has several advantages. However, high strength lightweight concretes have significant benefits over high strength normal weight concrete because of the reduction of dead load and the construction costs. Since last few decades, the lightweight concrete has attracted a considerable attention. The number of researchers have investigated the lightweight concretes by using several types of lightweight aggregates. However, not all types of lightweight aggregates are suitable for the production of high strength lightweight concrete (Zhang & Gjorv, 1990).

The OPS and OPBC are an alternative lightweight aggregate from the oil palm industry. Several researchers have investigated the properties of the lightweight concretes by using OPS as lightweight aggregate. However, the test results of time dependent properties of this type of LWC has shown that the most grades of this concrete cannot be utilized for the structural purposes. It is because of high drying shrinkage and consequently high creep value. Further, it was also observed that OPS concrete is very sensitive to the curing and a significant reduction in the compressive strength and other mechanical properties was observed in poor curing environment. Furthermore, for structural applications, there is not much information available on the structural behaviour of oil palm shell or oil-palm-boiler clinker concretes.

Based on the reasons mentioned above, the present research program was developed to investigate the mechanical properties and the structural performance of the high-strength lightweight concrete by using oil palm bio-products as a lightweight aggregate.

1.5 Objectives of the Research

The main objective of this research is to develop a sustainable high strength lightweight aggregate concrete to be used in structural elements using bio-product wastes from palm oil industry. The sub-objectives of this study are as followings:

1. To investigate the optimum contribution level of oil-palm-boiler clinker (OPBC) in oil palm shell (OPS) concrete to produce lightweight aggregate concrete for structural applications.
2. To study the mechanical properties of the high strength lightweight aggregate concrete made of blended of OPS and OPBC.
3. To investigate the drying shrinkage behaviour of lightweight concretes made of bio-product aggregates.
4. To evaluate the shear behaviour of reinforced concrete beams made of OPS and OPBC.

1.6 Scope of the Study

The experimental investigation consisted of conducting the physical and mechanical properties of all the aggregates namely oil palm shell (OPS), oil-palm-boiler clinker (OPBC), lightweight expanded clay aggregate (LECA) and the normal weight aggregate (NWA). After that different mix proportions of the lightweight, semi-lightweight and the normal weight concretes were prepared to identify the physical, mechanical, durability and drying shrinkage of the concretes. While, the structural performance involved shear behaviour of the lightweight reinforced concrete beams under flexure was carried out. The aim of this study was to produce a new type of lightweight concrete which can be applicable for the real-life structures. The role of different types of aggregates in the concrete was also considered under different curing regimes. In structural performance, the experimental program consisted of nine lightweight reinforced concrete beams. All the beams were designed to investigate the shear behaviour of the lightweight concrete beams containing oil palm by-products as lightweight aggregates. The main differences between the beams are the grade of concretes, type of aggregates, cement contents and fly ash to examine the effect on the shear and cracking behaviours, and the failure modes under flexural loading.

1.7 Thesis Layout

The results of this research investigation in the thesis have been planned to publish in various ISI indexed and refereed journals. Although, some of results were already published in several ISI indexed journals, however, still some of articles are still under process. This thesis is divided into ten chapters. Chapter one provides the basic introduction about the research area and specifies the research needs, objectives and the scope of work. A comprehensive critical review on the use of by-product materials based on the relevant topic is presented in chapter two. The literature provided in this chapter was already published as a review paper entitled "Oil-palm by-products as lightweight aggregate in concrete mixture - A review".

The mechanical properties of the prepared lightweight and semi-lightweight aggregate concretes by using blended coarse aggregates with their benefits and advantages were discussed and presented in chapters three, four and five. Among these, the chapters three and four were already published in ISI indexed journals, however, the fifth one is still under review in the ISI indexed journal.

The time-dependent deformation, drying shrinkage of the oil palm shell (OPS) and OPS-OPBC concretes and their comparisons with other types of concretes, effects of different types of coarse aggregates, and the validation of the experimental results with different prediction models are presented and discussed in chapters six, seven and eight. Among these, the article six already published in ISI indexed journal, whereas, seventh and eighth are submitted to the ISI indexed journals.

In structural performance, the shear behaviour of the oil-palm by-product lightweight concretes without shear reinforcement is presented in chapter nine. After some modifications, soon the article will be submitted to the ISI indexed journal. Finally, chapter ten presented the cumulative conclusions of the major findings of the present research investigation, as well as some future recommendations were also suggested.

CHAPTER 2: LITERATURE REVIEW

2.1 Introduction

Structural concrete is the most widely used construction material in the majority of civil engineering structures (Hosseini, Booshehrian, Delkash, Ghavami, & Zanjani, 2009). It has an excellent resistance to water and can be formed into a variety of shapes and sizes (Calkins, 2009). Nowadays, the concrete industry consumes 1.5 billion tonnes of cement, 10-12 billion tonnes of sand and rock together, and 1 billion tonnes of mixing water annually (Shafigh, Jumaat, Mahmud, & Alengaram, 2013). This means that a huge amount of raw materials and natural resources are being consumed for the production of concrete worldwide (Altwair & Kabir, 2010). Because of the huge amount of concrete being produced daily, even a small reduction in the environmental impact per tonne of concrete will result in considerable benefits to the environment (Silva, Brito, & Dhir, 2016). This reduction can be achieved by considering the composition of ingredients constituting structural concrete, most importantly, the composition of ordinary Portland cement (OPC).

The high demand for concrete in construction using normal weight aggregates (NWAs) has drastically reduced natural stone deposits and caused irreparable damage to the environment. As a result, recently, the emphasis on sustainable materials has intensified (Alengaram, Al-Muhit, & Jumaat, 2013). The growing need for sustainable development has motivated researchers to focus their investigation on the use and conversion of waste or recycled material into potential construction materials (Mo, Alengaram, Jumaat, & Yap, 2015).

Lightweight concrete (LWC) is not a new invention in concrete technology. It has been used since ancient times and is a most interesting field of research because of its advantages, such as better heat insulation, fire and frost resistance, sound absorption, superior anti-condensation properties and increased seismic damping (Shafigh et al., 2010). The most popular way of achieving LWC production is by using a lightweight aggregate (LWA) (Polat, Demirboga, & Karakoc, 2010). The LWAs are generally classified into two groups: natural and artificial. The main natural LWAs are pumice, diatomite, volcanic cinders, scoria and tuff (Neville & Brooks, 2008). The artificial aggregates are further classified into two groups, namely, the industrial by-products and modified naturally arising materials. The industrial by-product materials utilized as LWAs are sintered slate, sintered pulverized fuel ash, expanded or foamed blast furnace slag and colliery waste. The naturally arising materials that need further processing (produced by the application of high temperature), such as shale, expanded clay, slate, vermiculite and perlite, are also used as LWAs in the construction industry (Shafigh et al., 2010).

The industrial waste materials that are mostly used as lightweight aggregates (LWAs) are expanded slag, sintered pulverized fuel ash, and bed ash, which promote the use of sustainable materials (Chandra & Berntsson, 2002). LWAs have been used over a long period of time during which they have proven to be cost effective and fulfilled the purpose of providing both the structural stability and economic viability (Emdadi, Asim, Yarmo, & Shamsudin, 2014). The lower the weight, the more versatile are the structures (Zhang & Poon, 2015). Since second millennium, different types of natural and manufactured LWAs, such as foamed slag, diatomite, pumice, volcanic cinders, scoria, tuff, expanded clay, shale, slate, perlite and vermiculite and materials that occur as industrial by-products, such as sintered pulverized-fuel ash, sintered slate and colliery waste, and foamed or expanded blast-furnace slag have been used as construction materials (CEB/FIP, 1983; Mo, Alengaram, Jumaat, Liu, & Lim, 2016). Oil palm shell (OPS) and oil-palm-boiler clinker (OPBC) are used as an alternative LWA in tropical regimes and countries that have a palm oil industry (Mo, Alengaram, & Jumaat, 2014). Countries such as Malaysia, Indonesia, and Thailand, have vast palm oil plantations. In fact, these countries contributed to about 90% of the total world's palm oil production in the year 2009 (Islam, Mo, Alengaram, & Jumaat, 2016; Liu, Alengaram, Jumaat, & Mo, 2014). Malaysia contributes about 58% of the total world supply of palm oil, and is one of the top listed countries with huge resources of OPS (Ahmad et al., 2010). OPS and OPBC are produced in large quantities by oil mills. For instance, in Malaysia, it was estimated that over 4 million tonnes of OPS solid waste is produced annually (Nagaratnam, Rahman, Mirasa, Mannan, & Lame, 2015), however, only a fraction is used for the production of fuel and activated carbon (Sobuz, Hasan, Tamanna, & Islam, 2014). The OPBC is an aggregate extracted from boiler of palm oil mill with higher specific gravity and density compared to other lightweight aggregates such as OPS, coconut shell, expanded clay, and lytag. It was reported by Shi, Wu, Lv, & Wu (2015) that the utilization of these waste materials in concrete leads to sustainable concrete and reduces environmental impact from the manufacture of concrete using conventional materials. In Malaysia, the concept of using OPS as a LWA in producing LWC was introduced by Sallam & Abdullah (1985). The number of studies are available in the literature regarding the use OPS as an aggregate in concrete mixture. However, study on the use of OPBC in concrete is new and it needs further investigations.

The main objective of this study is to review the potential use of OPBC as a lightweight aggregate in concrete. A detailed study was conducted to identify the physical and chemical properties of OPBC and its properties were compared with other LWAs. In addition, the effect of using OPBC in concrete mixture on the mechanical properties of concrete is discussed. In light of the available literature, the authors believe that significant achievements can be attained by analyzing and summarizing the properties of OPBC. Furthermore, new subjects for the research are identified for researchers to explore innovative LWC based on financial and environmental design factors.

2.2 Sustainable Development

Sustainable development is the major issue in the world these days. It meets the needs of the present without compromising the ability of future generations to meet their own needs. It involves the practices which would eventually produce high efficiency products for the benefit of the ecosystem and the mankind (Brundtland, 1987). The sustainable development involves a progressive transformation of economy and society. The major objective of this development is the satisfaction of human aspirations and needs (Drexhage & Murphy, 2010). To fulfil the basic requirements in the human life, demands are increasing regularly, including food, fresh air, clean water, clothing, shelter, rapid and safe transport of people and goods, waste disposal, industrial and residential buildings, and the sources of energy (Karim, Zain, Jamil, & Lai, 2011). United Nations (UN) provides a comprehensive agenda 21 on actions to be taken locally, nationally and globally by major groups, governments and organizations in every area in which humans influence the environment. The task is to balance economic development with social and environmental objectives (United Nations, 1993). The palm oil industry has nevertheless generated a million tonnes of waste every year and its sustainability covers the whole manufacturing system. The sustainability of the palm oil industry mainly focuses the countries and international acts, standards and regulations, life cycle assessment (LCA), reverse logistics, GHG emissions, new degradation method, zero waste, energy generation, renewable energy, waste treatment systems, eco-label and bio-energy, food products and consumption, optimization processes, clean production, energy efficiency, and process improvement (Abdullah, Mahmood, Fauadi, Rahman, & Ahmad, 2015). The sustainable manufacturing process of the palm oil industry is shown in Figure 2.1.

In fact, the entire world accepted that concrete performs outstanding responsibilities in the modern construction industry such as infrastructures, urbanization and industrialization for the growing population (Blodgett, 2004). Beside this, it is important to mention that the concrete industry today is one of the largest consumer of natural resources (Mefteh, Kebaili, Oucief, Berredjem, & Nourredine Arabi, 2013). For these reasons, sustainable concrete is one of the major topics in concrete industry all over the world and its main objectives including: reduce the amount of pollution and carbon dioxide (CO_2) gas, development of low energy resources, more efficient use of waste materials, long lasting, flexible buildings and structures and developing the thermal mass of concrete in a structure to reduce energy demand (Bravo, Brito, Pontes, & Evangelista, 2015; Rodrigues, Carvalho, Evangelista, & Brito, 2013).

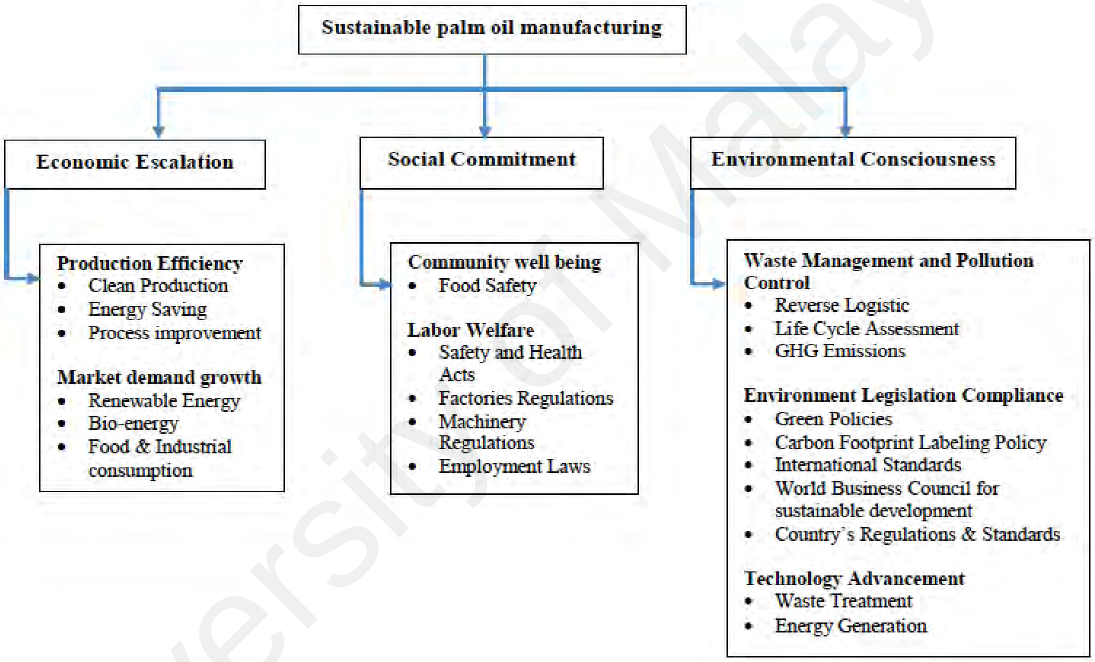


Figure 2.1: Sustainability of the manufacturing process of the palm oil industry

2.2.1 Cleaner Production

The term cleaner production can be defined as a continuous application of an integrated preventive environmental strategy applied to products, processes and services to increase overall efficiency and reduce risks to humans and the environment (Drexhage & Murphy, 2010; United Nations, 1993). The goal of cleaner production is to improve the eco-efficiency in companies by implementation of technical actions and by reducing the negative effects to the environment. The cleaner production method has several benefits including improving environmental situation, increasing economical benefits, increasing productivity, continuous environmental improvement and gaining competitive advantage (Drexhage & Murphy, 2010). To avoid or to overcome the barriers and to guarantee a successful implementation, cleaner production calls for an organized approach (UNEP, 2013). The cleaner production assessment also can be described as consisting of four basic steps, as shown in Figure 2.2.

The agro-based industries play a major role in the economy of the Malaysia from last few decades, and the products from this industry generated a huge amount of wastes that needs suitable disposal (Kanadasan & Razak, 2014a). Oil palm factories generate various types of wastes, including, OPS, fibres, OPBC, palm oil mill effluent (POME) and empty fruit bunches (EFB). Improper management of these wastes could result the environmental pollution. The 3R (Reduce, Reuse, Recycle) concept could be more beneficial to save the environment besides supporting the sustainability of certain industries (Kanadasan & Razak, 2014b). For the clean production, the oil palm industry is following the following stages in manufacturing process and procurement of materials: 1) Fruit contamination control, 2) the increase of free fatty acids control, 3) best operating procedures application, 4) wasted oil collection, 5) standard operating procedures assertion, 6) empty fruit bunches utilization, 7) water use efficiency improvement, and 8) liquid waste utilization for producing bio-gas. These aspects of implementation of the cleaner production was then analyzed and verified by experts using Analytical Hierarchy Process (AHP) (Ling, 2011). Usage of solid wastes such as OPS and OPBC as aggregate in concrete mixture can be considered as one of the environmental benefits and green production that will be recognized by most of the sustainability rating systems such as the United States Green Building Council's (USGBC) which is Leadership in Energy and Environmental Design (LEED) (Leed, 2009).

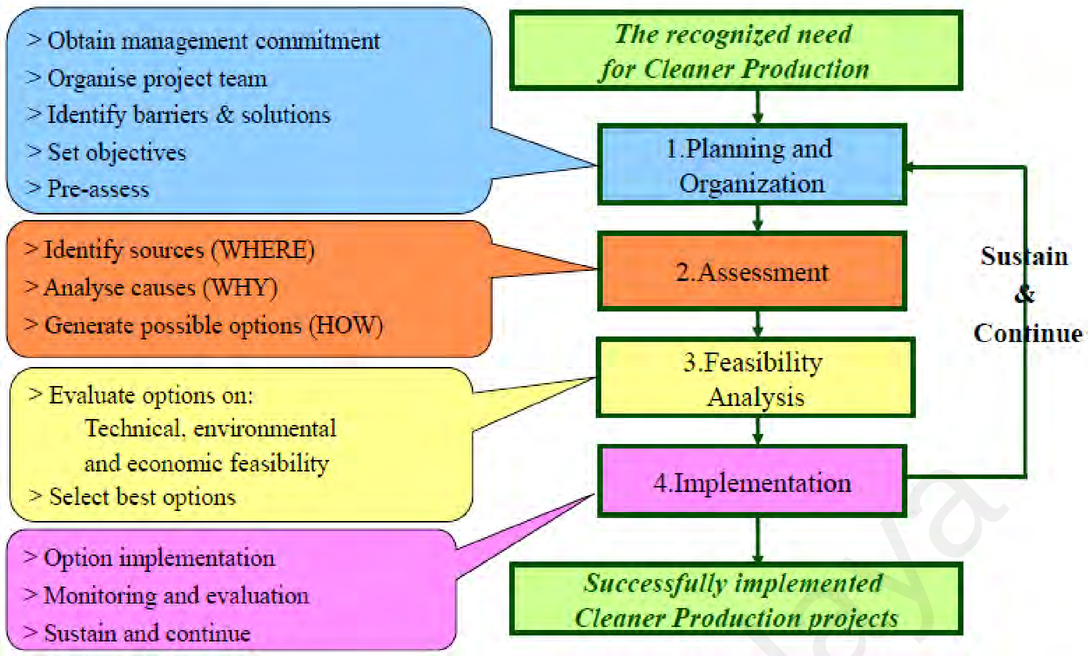


Figure 2.2: Methodology of the cleaner production

Source: (UNEP, 2013)

2.2.2 Environmental Sustainability Assessment

The environmental sustainability of a products can be ensured periodically by carrying out Environment Impact Assessment (EIA) over its life cycle. During such assessments, several factors were critically measured including efficient design of products and processes, selection of raw or natural materials (resources), recycling and reuse of the products, evaluation and assessment processes for waste material usage and wastes generation, development in energy efficiency of the system etc. The environmental impact assessment is a systematic method for identifying, evaluating and analyzing the major environmental effects of a production process throughout the life cycle of the product. The Life Cycle Assessment (LCA) is an important tool in EIA system that can be used to identify, evaluate and analyze the major environmental issues of the production process throughout the life cycle of the product (Heijungs, Huppes, & Guinee, 2010; ISO-14040, 2006; Rebitzer et al., 2004). The detailed framework of the LCA for any individual project depends on three most important steps as shown in Figure 2.3.

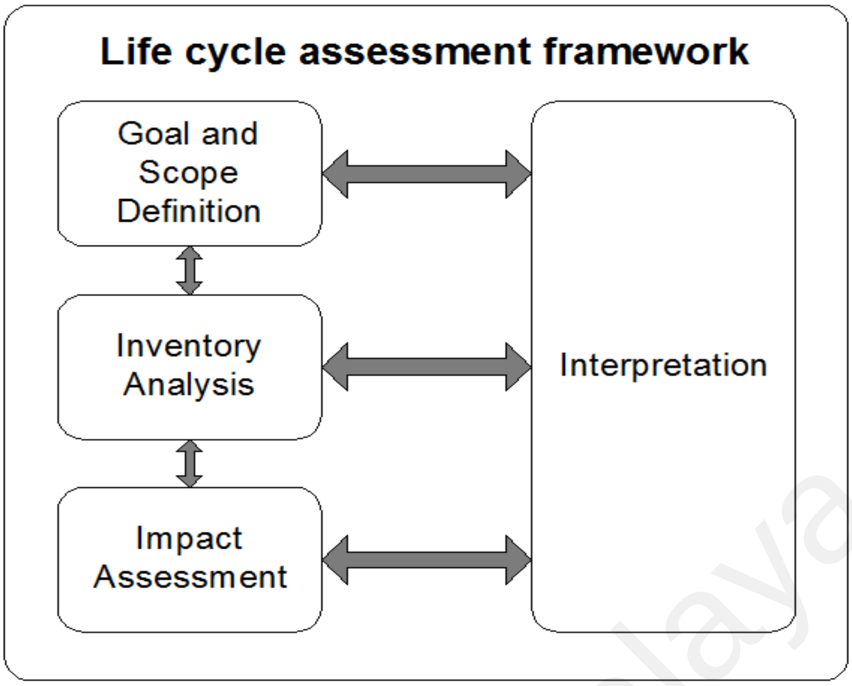


Figure 2.3: The stages of Life cycle assessment

Source: (ISO-14040, 2006)

During the production process of the oil palm industry, a huge of black smoke released which contains various pollutant materials and gases such as carbon monoxide (CO), nitrogen dioxide (NO_2) and Sulphur dioxide (SO_2) which have significant effect to environment and creatures (Lee & Ofori-Boateng, 2013). The total environmental impact of the manufacturing process of oil palm industry was assessed by using LCA as defined by (ISO-14040, 2006), it works in different phases.

The first phase is to notify the goal and the scope. The LCA assessment notify five main impact categories: 1) Non-renewable energy requirement, 2) Eutrophication potential, 3) Global warming impact, 4) Acidification impact, and 5) Occupation and land-use change impact on the ecosystem and useful quality of ecosystem. LCA performed for the whole manufacturing system including cultivation, oil extraction and refining, biodiesel production and transportation, infrastructure and maintenance, and byproducts at all processing stages (Achten, Vandenbempt, Almeida, Mathijs, & Muys, 2010). The major environmental impact categories with the environmental sustainability for the production process of waste materials from the oil palm industry are mentioned in Table 2.1.

The second phase is the life cycle inventory (LCI). In this phase, the data will be collected through different sources, either can be descriptive data or factory specific data and expert interviews. In addition, the field data on biodiversity, soil, vegetation structure, water balance and biomass necessary for the impact of land-use, were collected from the oil palm industry. From this specific information, the amount of carbon can also be computed (Achten et al., 2010; Eggleston, Buendia, Miwa, Nagara, & Tanabe, 2006). Continue to this, the production system was carried out which performs two main functions during the refinery process. After bleaching, cleaning, and separating the free fatty acids, the stearin fraction was separated from the olein fraction. Then the stearin fraction was sent to bio-diesel production and olein fraction is sold as cooking oil. After that the product output and fossil reference system was performed, in that all the products and by-products of the palm oil manufacturing system should be substituted in the fossil reference system. The substitutions reflect to the local reference system, where crude palm oil is used as kitchen oil. Other by-products of the biodiesel system are palm kernel oil which is sold as fine oil for the cosmetic industry, palm kernel meal, that can be used as local animal feeds (Lam, Lee, & Mohamed, 2009; Yee, Tan, Abdullah, & Lee, 2009).

The third phase is the evaluation of the environmental impacts, the overview of the basic calculation methods are provided here as; the impact calculation for nonrenewable energy requirement is the sum of fossil energy requirement throughout the life cycle. The global warming potential can be computed as the sum greenhouse gas emissions throughout the life cycle. Acidification potential is the sum of NH_3, NOx, and SOx emissions throughout the life cycle, whereas, Eutrophication potential is the sum of nitrogen emissions and flows to water ways or ground water. The land-use impact depends on land-use change (LUC) and land occupation (LO) on ecosystem quality (EQ) (Achten et al., 2010).

Table 2.1: Environmental impact categories with the environmental sustainability for the production process of waste materials from the oil palm industry

S. No	Impact category	Characterization factors
1.	Climate change	Greenhouse gas (GHG), ocean acidification, global warming
2.	Biodiversity loss	Wildlife species destruction, endangered species, pollinator decline
3.	Water quality change	Waste water pollution, eco-toxicity, marine pollution, urban runoffs
4.	Air quality change	Smog, air pollution, tropospheric ozone, volatile organic compound,
5.	Land use and degradation	Land pollution, desertification, overgrazing, habitat destruction
6.	Soil degradation	Soil erosion, soil contamination, soil salinity
7.	Energy use	Energy balance, renewable energy use, efficiency in energy use
8.	Resource use and depletion	Over use of natural resources, deforestation, illegal logging
9.	Chemical emissions	Waste disposal incidence, herbicides drifts, landfills, incineration
10.	Ozone depletion	Ultraviolet exposure, chlorofluorocarbons
11.	Nuclear emissions	Nuclear radiations, nuclear weapons, radioactive waste emissions
12.	Human toxicity	Respiratory organics and inorganics, heavy metals

Source: (Lee & Ofori-Boateng, 2013)

2.3 Oil-palm-boiler Clinker (OPBC) as Aggregate

2.3.1 Origin of OPBC

Oil palm is a widely-cultivated oil bearing tropical palm tree that originated from West Africa and was first illustrated by Nicholas Jacquin in 1763 (Lee & Ofori-Boateng, 2013). Oil palm milling is the process of extracting oil palm from fresh fruit bunches (FFBs) and producing crude palm oil and palm kernels (Ahmad, Hilton, & Noor, 2007b). The global palm oil industry generates over 190 million tonnes of waste in the form of solid and liquid residues. Out of this, only about 10% is utilized commercially for value-added bio products like bio fertilizers. The by-product or waste is in the form of empty fruit bunches (EFBs), fibres, shells, trunk, fronds and palm oil mill effluent (POME). POME is the largest amount of liquid waste, which, basically, consists of 95% water, 4% residual oil and 1% suspended solid material (Lee & Ofori-Boateng, 2013). Figure 2.4 shows the quantity of solid waste bio products of oil palm industries in Malaysia in million tonnes per year.

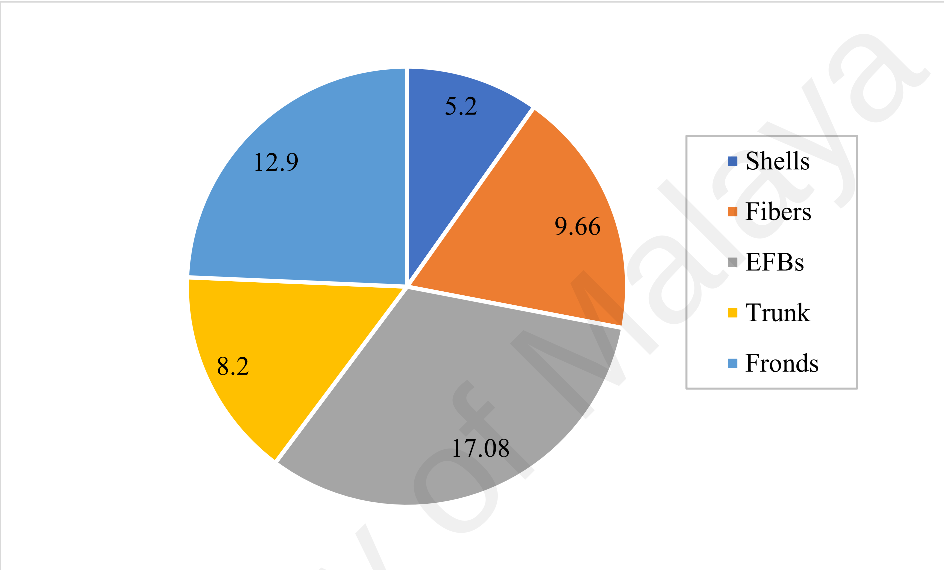


Figure 2.4: Oil palm bio-products in Malaysia (millions of tons per year)
Source: (Hosseini & Wahid, 2014)

During the oil palm extraction process, as shown in Figure 2.5, the shells and fibres are the ingredients of the oil palm used as fuel for firing the furnace of the mill to heat up the boiler. After about 3-4 h of burning at a temperature of 400 °C, the ashes of the fibres and shells are combined, possibly with some other impurities, to form a by-product, which is locally known as 'boiler stone'. It is also referred to as oil-palm-boiler clinker (OPBC) (Soleymani, 2012). Oil palm mills in Malaysia use 96-98% of fibres and 60-80% of shells as their source of fuel for the boilers to generate electricity and steam for oil palm extraction (Kong, Loh, Bachmann, Rahim, & Salimon, 2014). MHES Asia Sdn. Bhd. is a biomass power plant located in Beau, Negeri Sembilan, which has a 13-MW capacity and uses EFBs as a fuel (Ludin et al., 2004; Shuit, Tan, Lee, & Kamaruddin, 2009). The stipulated annual cost saving in running industrial boilers when biomass and fossil fuel are applied simultaneously is around US$270,000 based on 80% diesel and 20% biomass consumption, and US$630,000 in the case of 50% diesel and 50% biomass consumption (Faaij, 2006).

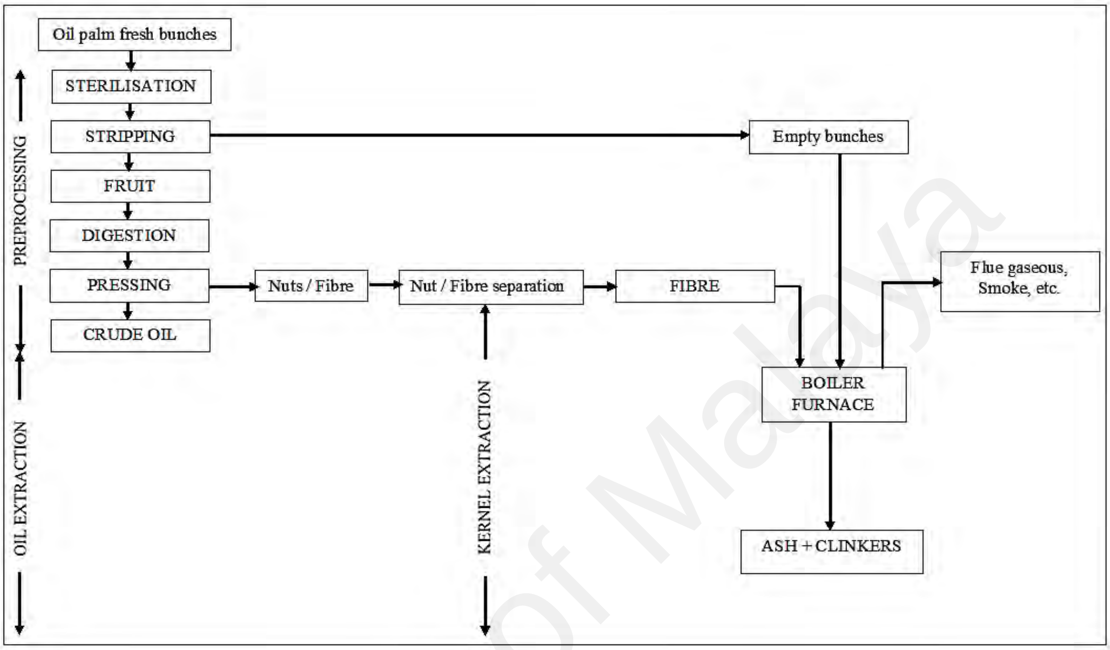


Figure 2.5: Chart of extraction process of oil-palm-boiler clinkers

2.3.2 Physical Properties of OPBC

The mechanical properties of OPBC concrete (OPBCC) depend on the physical properties of OPBC aggregate. In this study, the investigated physical properties are specific gravity, shape, thickness, surface texture, loose and compacted bulk densities, air and moisture content, porosity and water absorption. These properties are compared with the properties of normal weight aggregates (NWAs).

2.3.2.1 Specific gravity

The specific gravity of a material is the ratio of the density of that particular material and that of water (Ahmad, Hilton, & Noor, 2007a). Researchers generally prefer OPBC both in the form of fine and coarse aggregates (Abdullahi, Al-Mattarneh, Abu Hassan, Hassan, & Mohammed, 2008; Ahmad et al., 2007a, 2007b; Ahmad, Noor, & Adnan, 2008; Kamaruddin, 1991; Mohammed, Foo, & Abdullahi, 2014; Mohammed, Hossain, Foo, & Abdullahi, 2013; Roslli & Mohamed, 2002; Zakaria, 1986). From the reports (Ahmad et al., 2007a; Alengaram et al., 2013; Mohammed et al., 2014), it can be seen that although OPBC has varying specific gravity values they never exceeded the value of the specific gravity of NWA (Table 2.2).

The range of specific gravity for OPBC is 1.7 to 2.2. The lowest value of 1.7 for the specific gravity of fine OPBC was reported by Zakaria (1986), while the researchers (Ahmad et al., 2007a; Chan & Robani, 2008; Robani & Chee, 2009) reported the highest value of 2.2 for the specific gravity of fine OPBC. The lowest proposed value of specific gravity for coarse OPBC is 1.7 (Mannan & Neglo, 2010), while Robani & Chee (2009) and Chan & Robani (2008) reported the highest value of specific gravity for coarse OPBC as 2.2. The main reason for such a variety of specific gravities may be due to the different sources. The values for the specific gravity of other artificial lightweight aggregates, such as Lytag and LECA, range from 0.8 to 0.9, and natural LWA such as pumice, is found in the range of 1.3 to 1.7 (Hemmings, Cornelius, Yuran, & Milton, 2009). However, the specific gravity of OPBC aggregates is 15 to 35 % less than the normal weight aggregates, 47 to 53% higher than the artificial LWAs and 24% higher than natural LWAs like pumice, diatomite and volcanic cinders.

Table 2.2: Physical properties of OPBC aggregate

Size of aggregate	Specific gravity	Loose bulk density (kg/m^3)	Compacted bulk density (kg/m^3)	Moisture content (%)	24 h Water absorption (%)	References
Fine Coarse	1.8 1.7	1120 790	-	-	14.3 5.4	(Abdullahi et al., 2008)
Fine Coarse	2.0 1.8	1120 780	-	0.11 0.07	26.4 4.4	(Mohammed, Al-Ganad, & Abdullahi, 2011)
Fine Coarse	1.97 1.73	-	-	0.5 ± 0.25 1 ± 0.5	10 ± 5 3 ± 2	(Kanadasan & Razak, 2015)
Coarse	1.7	-	800	-	2.7	(Mannan & Neglo, 2010)
Fine Coarse	- 1.73	-	1080 827	-	7.5 3.0	(Roslli & Mohamed, 2002)
Fine Coarse	2.2 1.8	- 740	1040 860	-	- 4.6	(Ahmad et al., 2007b)
Fine Coarse	1.7 1.95	-	1080 815	-	-	(Zakaria, 1986)
Coarse	2.2	-	860	0.08	4.6	(Ahmad et al., 2008)
Fine Coarse	2.2 1.7	-	-	0.5 ± 0.25 1 ± 0.5	10 ± 5 3 ± 2	(Kanadasan & Razak, 2014a, 2014b)

2.3.2.2 Sieve analysis of OPBC

The sieve analysis, also known as the gradation test, is a basic essential test for all aggregate technicians. The sieve analysis determines the gradation (the distribution of aggregate particles, by size, within a given sample) in order to determine compliance with the design, production control requirements and verification specifications. OPBC is available as both fine and coarse aggregate and is a well-graded and promising material for use in concrete work, as shown in Figure 2.6 (Abdullahi et al., 2008; Mahmud, Jumaat, & Alengaram, 2009; Mannan & Neglo, 2010).

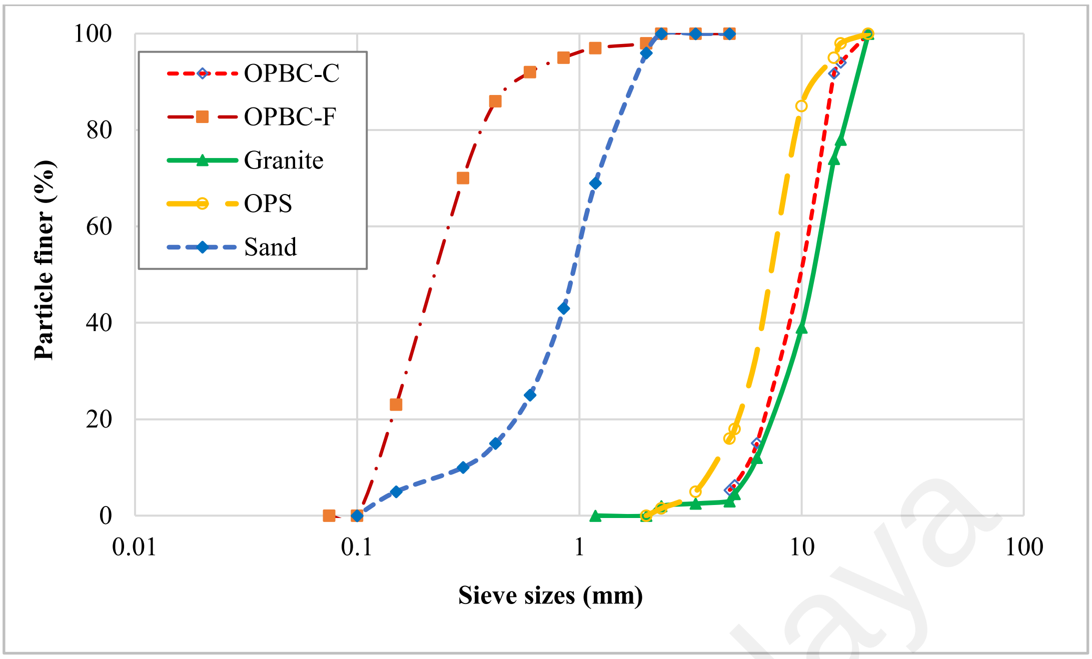


Figure 2.6: Particle size distribution of the OPBC compared with other LWA reported in the literature

Source: (Abdullahi et al., 2008; Alengaram et al., 2013; Mahmud et al., 2009; Mannan & Neglo, 2010)

2.3.2.3 Shape thickness and texture

When OPBC is withdrawn from the furnace, it is found in lumps of irregular shapes with a size varying from 150 to 225 mm, as shown in Figure 2.7. OPBC is whitish grey in colour and has the appearance of a porous stone, a feature that contributes to its lightweightness. It is usually flaky and irregular in shape, with rough and spiky broken edges (Aslam, Shafigh, Jumaat, & Lachemi, 2016). However, OPBC with a nominal size of 20 mm is used as a coarse aggregate, and, for fine aggregate, a size below 4.75 mm is preferred (Ahmad et al., 2008).

(a) (b)

Figure 2.7: OPBC aggregates: a) OPBC big lumps, b) Prepared OPBC for concrete

Source: (Abdullahi et al., 2008; Alengaram et al., 2013; Mahmud et al., 2009; Mannan & Neglo, 2010)

2.3.2.4 Bulk density

The bulk density estimates how tightly the aggregates are packed together and shows the approximate size distribution and shape of the particles (Ahmad et al., 2007b). The loose and compacted bulk densities of OPBC aggregate vary within the different ranges, as shown in Table 2.2. The compacted bulk density for fine OPBC varied in the range of 860 to 1080 kg/m^3, while the loose and compacted bulk densities of coarse OPBC ranged from 740 to 790 kg/m^3 and 800 to 840 kg/m^3 (Abdullahi et al., 2008; Ahmad et al., 2007a, 2007b; Chan & Robani, 2008; Mannan & Neglo, 2010; Mohammed, Al-Ganad, et al., 2011; Mohammed et al., 2014; Mohammed, Hossain, Foo, & Abdullahi, 2011; Mohammed et al., 2013; Robani & Chee, 2009). For structural satisfaction, the density of the aggregates ranged between 700 and 1400 kg/m^3 (Ahmad et al., 2007b).

The bulk density of fine OPBC is 35 to 43% lower than normal sand, and the density of coarse OPBC is 40 to 45% lower than conventional coarse aggregate. However, compared to other types of lightweight aggregate originating from agricultural waste, namely oil palm shell and coconut shell, the density of OPBC is 23 to 30% higher. The bulk density of coarse OPBC is 6 to 46% higher than the artificial LWAs, namely LECA and Lytag, and 6 to 66% higher than natural LWAs like pumice, diatomite and volcanic cinders.

2.3.2.5 Water absorption

OPBC aggregate is a porous material and will absorb huge amounts of water compared to the normal weight aggregate (Ahmad et al., 2007a). The 24-h water absorption for fine OPBC aggregates ranges from 4.7 to 26.5% and for coarse it found in the range of 1.8 to 5.4%. Although, OPBC aggregate has a high-water absorption, it was reported that pumice aggregate has a higher water absorption of approximately 37% (Hossain & Khandaker, 2004). For the purpose of concrete mix design, it is very important to specify the quantity of water absorption that is required for the aggregates, otherwise, it will create a problem of workability and consistency of the concrete (Ahmad et al., 2007b). The high-water absorption of OPBC aggregate can be valuable for the resultant hardened concrete. It is reported that LWC with a porous aggregate is less sensitive to poor curing compared to NWC, especially in the early ages due to the internal water supply placed in the pores of LWAs (Mohammed et al., 2014).

The utilization of OPBC particles in the geo-polymeric samples to observe the water absorption was carried out by several researchers (Nazari, Riahi, & Bagheri, 2012; Sanjayan, Nazari, Chen, & Nguyen, 2015). The results showed that using the OPBC particles results in a good resistance to water absorption with respect to the OPBC-free specimens at early ages which make them appropriate for weightless applications. They used Fuzzy logic and ANFIS techniques to prediction the experimental test results (Nazari, 2012; Nazari & Khalaj, 2012). It was found that by using of these techniques the experimental results can be predicted by 90-99% reliability.

The water content of the aggregates mostly depends on the humidity of the atmosphere in which the aggregates are stored. If the aggregates have been stored in the laboratory, they will not be exposed to rain, which creates moisture not only on the surface but also inside the aggregates as the pores will absorb water inside the particles. This amount of moisture is of significant importance while the concrete mix design is carried out as it is impossible to dry the bulk quantity of the aggregates on site (Ahmad et al., 2007b). It was found that the free moisture content of OPBC varied between 0.05 to 1%, and the moisture content of the NWA was typically found to be in the range of 0.5 to 1% (Alengaram et al., 2013).

2.3.3 Comparison of the Chemical Composition of OPBC with Other LWAs

As with the physical properties of the materials, it is also very important to know the chemical properties of OPBC to ensure that when using the binder it behaves properly. In material selection, the use of OPS and OPBC, either alone or in combination, is very effective in reducing the permeability of the concrete (Bremner & Holm, 1995; Holm & Bremner, 1991). Mostly, in LWC, the lower permeability is expected due to the improved contact zone between the LWA and the binder paste. The improved contact zone is due to internal curing and to the vesicular nature of the aggregate, which enables the paste to seep into the LWA particles for a better bond, and the pozzolanic nature of the aggregate surface, which enables a chemical bond between the aggregate and the paste (CEB/FIP-83, 1983). In LWC, the expansive products caused in these chemical reactions, if they ever occur, can move into the pores of LWA, minimizing the distress (Bremner & Holm, 1995). The chemical composition of OPBC aggregate compared with other LWA aggregates is mentioned in Table 2.3. As can be seen in this table, the chemical composition of OPBC is almost similar to natural lightweight aggregate like pumice and artificial lightweight aggregates such as lightweight expanded clay aggregate (LECA) and Lytag. In contrast to these lightweight aggregates, OPS has much different chemical compositions showing that it is a volatile organic (agricultural waste) material. Therefore, it is expected that concretes containing OPBC have better performance compared to OPS concrete particularly at elevated temperature.

Table 2.3: Chemical properties of OPBC aggregates compared with other LWAs

Element		Concentration (%)				
		OPBC	OPS	Pumice	LECA*	Lytag
Calcium Oxide	CaO	2.3-8.2	0.08	1.0-2.0	2.0-2.5	3.0-4.0
Silica dioxide	SiO_2	59.6-81.8	0.01	60.0-75.0	62.0-66.0	50.0-53.0
Ferric Oxide	Fe_2O_3	4.62-5.2	0.03	1.0-7.0	7.0-9.0	5.0-6.0
Sulphur trioxide	SO_3	0.73	-	0.14	1.0-16.0	0.3
Aluminum Oxide	Al_2O_3	3.5-3.7	0.13	13.0-17.0	0.2-16.0	23.0-25.0
Magnesium Oxide	MgO	1.2-5.0	0.03	1.0-2.0	1.0-4.0	2.8-3.0
Phosphorous pentoxide	P_2O_5	0.8-5.3	-	-	0.21	-
Potassium Oxide	K_2O	4.6-11.6	0.00	7.0-8.0	2.0-3.5	0.2
Titanium dioxide	TiO_2	0.2	-	-	-	-
Natrium Oxide	Na_2O	0.1-0.3	0.00	3.0-5.0	0.7-2.0	0.3
Manganese trioxide	Mn_2O_3	-	-	-	-	-
Manganese Oxide	MnO	-	-	-	0.14	-
-	LOI	-	-	-	0.84	-
Ash	-	-	1.53	-	-	-
Nitrogen	N	-	0.41	-	-	-
Sulphur	S	-	0.00	-	-	-
Chloride	Cl^-	-	0.00	-	-	-
Loss on ignition	-	-	98.5	1.52	-	3.1

Source: (Ahmmad, Jumaat, Bahri, & Islam, 2014; Arioz & Karasu, 2008; Hemmings et al., 2009; Hossain & Khandaker, 2004; Robani & Chee, 2009; Shafigh, Mahmoud, Razavi, & Kobraei, 2011; Shafigh et al., 2010)

2.3.4 Mechanical Properties of OPBC Compared with Other LWAs

The mechanical properties of OPBC aggregate, such as Los Angeles abrasion test, and impact and crushing values, have been reported by many researchers. It has been reported that the structure of OPBC aggregate and its rate of water absorption can significantly affect the mechanical properties of the LWC (Ahmad et al., 2008). The density and strength of the structural lightweight concrete depend on the mechanical properties of OPBC aggregate (Mohammed et al., 2013). The mechanical properties of various LWAs compared with OPBC aggregate are shown in Table 2.4. The crushing value of the aggregates give a relative measure of the resistance of an aggregate to crushing under a gradually applied compressive load. The crushing value of aggregates is restricted to 30% for concrete used for roads and pavements, while 45% may be permitted for other structures (Mehta & Monteiro, 2006). The crushing value for OPBC aggregate is 6 to 10% lower than for crushed granite, and can be used for all types of structures.

In concrete, the toughness of the aggregates is usually considered as the resistance of the material to failure by impact. IS-283 (1970) specifies that the impact value of aggregates shall not exceed 45% by weight for aggregates used for concrete other than for the wearing surface, and 30% by weight for concrete for the wearing surface (Mehta & Monteiro, 2006; Neville & Brooks, 2008). With respect to impact and crushing resistance, the resistance to wear is also an important factor for the aggregates. Generally, the abrasion resistance of LWA is inferior to that of NWA due to the lower stiffness of LWA (Alengaram et al., 2013). The abrasion value should not be more than 30% for the wearing surface, and not more than 50% for concrete other than for the wearing surface (Neville & Brooks, 2008). As can be seen in Table 2.4, the abrasion value for OPBC aggregate is around 8 to 26% higher than for crushed granite. It was observed that all the mechanical properties for OPBC aggregate were found to be in suitable ranges, and, can be used in the construction industry as aggregate. Table 2.4 shows that the crushing, impact and abrasion values of OPS and coconut shells are much lower than OPBC and granite aggregates. The main reason for this difference is that these shells are similar to wood and are relatively ductile materials while OPBC like granite aggregate is similar to stone with brittle behaviour.

Table 2.4: Mechanical properties of OPBC compared with other LW & NW aggregates

Properties	Agricultural solid wastes			Crushed granite
	OPBC	Oil Palm Shell	Coconut shell	
Aggregate Crushing value (%)	18.0-18.8	5.0-10.0	2.6	20.0
Aggregate Impact value (%)	25.0-38.6	4.0-8.0	8.2	13.0-17.0
Los Angeles abrasion value (%)	27.1	3.0-5.0	1.6	20.0-25.0

Source: (Ahmad et al., 2007b; Ahmad et al., 2008; Mohammed et al., 2014; Mohammed, Hossain, et al., 2011; Shafigh, Mahmud, Jumaat, Ahmmad, & Bahri, 2014)

2.4 Mix Design for OPBC Concrete

In well-proportioned mixtures, the amount of cement and strength relationship are fairly constant for a particular type or between one type and another. Therefore, researchers prefer different trial mixtures with varying cement content, OPBC as fine and coarse aggregates as well as other normal weight aggregates to develop the required range of compressive strengths. From the literature, the selected mix proportions, and fresh and hardened properties of OPBC concrete are shown in Table 2.5. Several researchers conducted a series of experiments using cement, and OPBC as fine and coarse aggregates to produce a structural lightweight aggregate concrete. It was reported that, without any admixture, OPBC aggregate gives good workability of the concrete. The slump value was found to be in the range of 45 to 190 mm while the 28-day compressive strength ranged between 17 and 47 MPa (Abdullahi et al., 2008; Mohammed et al., 2014). The several trial mixes to determine the suitable mix proportions for OPBC lightweight concrete was examined by Mannan and Neglo (2010); they preferred three different mix design methods. In all the mix proportions, the materials used were cement, river sand and OPBC as the coarse aggregate, together with a super-plasticizer (SP). The slump value of the mixes was in the range of 40-100 mm with the 28-day compressive strength and dry density in the range of 27-36 MPa and 1845-1980 kg/m^3, respectively. They also reported that the addition of SP did not increase the workability of OPBC concrete, while Zakaria (1986) used the same materials without any admixture and reported very low slump values and a 28-day compressive strength range of 15 to 28 MPa. The effect of SP in OPBC concrete was previously a controversial issue because it was totally a new waste material. For comparison with NWC, Ahmad et al. (2007a) prepared five different mix trials with one normal weight concrete and four LWC mixes. The NWC mix design was based on the method proposed by the Department of Environment (DoE), while the LWC series was designed using the Federation Internationale de la Precontrainte (FIP) method.

The four mixes of LWC were further divided into two sets. The first set contained two mixes; one mix was prepared using normal sand as the fine aggregate and OPBC as the coarse aggregate. The second mix contained OPBC as both the fine and coarse aggregates. It was observed that the mix prepared by using OPBC as the coarse and fine aggregates gave an 18% lower compressive strength compared to the other trial mixes. The second set was prepared by using the same aggregates as in the first set; however, in this set, the cement was replaced with 10% fly ash. It was observed that the mix was prepared by using OPBC as the coarse and fine aggregates and fly ash gave a 12% lower compressive strength compared to the other trial mixes. Due to the porous nature of OPBC aggregate, they preferred the use of an admixture to compensate the workability issue and to achieve a stronger and more durable concrete. Finally, they concluded that OPBC concrete with coarse OPBC, fine sand and 10% fly ash replacement in cement content produced higher compressive and tensile strengths than concrete having 100% OPBC aggregates (coarse and fine).

The comparative behaviour of normal and lightweight concretes using normal and OPBC aggregates was studied by Hassan, Al-Mattarneh, Abdullahi, Mohamed, & Mustafa (2008). A normal weight concrete mix was considered as the control mix, and, in the other four mixes, the conventional aggregates were replaced in increments of 25% with OPBC fine and coarse aggregates. The slump value and compressive strength of all the mixes were found to be in the range of 0 to 70 mm and 26 to 42 MPa. It was observed that the compressive strength of OPBC concrete increased up to 5% compared to the control mix, through replacement with 25% OPBC coarse and fine aggregates. However, a 35% reduction in the compressive strength and zero slump value compared to control mix were observed through the 100% replacement with OPBC coarse and fine aggregates. The reduction in the slump value and the compressive strength of OPBC concrete was due to the high porosity and water absorption of OPBC aggregate. While, the durability properties of foamed OPBC concrete was investigated by (Chandran, 2010). They reported that the foamed concrete incorporating OPBC aggregates has better chemical resistance compared to conventional concrete.

The behaviour of the OPS concrete by replacing the normal sand with fine OPBC aggregate was conducted by Shafigh et al. (2014). The OPS concrete with normal sand was considered as the control concrete, and, in other mixes, the river sand was replaced with fine OPBC aggregate in increment of 12.5% up to 50% replacement of sand. The slump value and compressive strength for all OPBC-OPS concrete mixes were found in the range of 30 to 76 mm and 32 to 37 MPa. While, Kanadasan & Razak (2014b) used the particle packing concept to investigate the fresh and hardened properties of self-compacting concrete (SCC) incorporating waste product of oil-palm-boiler clinker aggregate. The actual packing level of aggregate and paste volume were integrated into the method. The results indicated that the mixture design could be employed not only for oil-palm-boiler clinker but also for a variety of combinations of aggregates. A study conducted from Aslam, Shafigh, & Jumaat (2015) revealed that OPBC coarse aggregate can be used as a partial replacement with OPS in OPS lightweight concrete. The advantages of using OPBC in OPS concrete were significant increase in compressive strength and efficiency factor (compressive strength to weight ratio).

2.5 Physical Properties of OPBC Concrete

2.5.1 Workability

One of the standard tests to check the workability of the concrete is the slump test, which measures the consistency of concrete according to ACI-116R (2000). It is very beneficial to manipulate the variations in the uniformity of the given mix proportions (Neville, 2008). Similar to NWC, the slump value of LWAC also increased with an increase in the water cement ratio; as shown in Table 2.5. It was reported by Shafigh et al. (2014) that OPBC, when used as a fine aggregate, it can absorb more water, even three times to the normal weight sand. Therefore, it is expected that by increasing the amount of OPBC aggregate and reducing the amount of normal sand in the concrete mixture, the slump value decreases. Abdullahi et al. (2008) found that the slump ranges varied from 45 to 190 mm without using any admixture and posited that concrete using OPBC would give good workability. After many trials, Mannan & Neglo (2010) obtained slump values ranging between 40 and 100 mm with the super-plasticizer and having a compressive strength up to 33 MPa. However, Ahmad et al. (2007a) used different types of aggregate, such as normal sand as the fine aggregate and OPBC as both the fine and coarse aggregates with the admixture. The slump value of concrete using OPBC as the fine and coarse aggregates ranged from 85 to 105 mm with a compressive strength of 34 to 42 MPa. The concrete casted with OPBC and fly ash had a slump value ranging between 100 to 125 mm, and a compressive strength between 36 to 42 MPa. They demonstrated that the 10% cement replacement by fly ash could even increase the compressive strength of OPBC concrete. They further reported that by incorporating a small percentage of admixture, a higher slump value could be achieved. They demonstrated that high range water reducing admixtures are capable of dispersing cement grains that lead towards a high slump value resulting in high workability. The detailed study was conducted using OPBC as the coarse aggregate and natural sand as the fine aggregate to fabricate LWC (Zakaria, 1986). The observed slump value was very low, varying from 1 to 15 mm with a compressive strength up to 28 MPa. However, this does not necessarily mean that the low slump ensures lower compressive strength. Hassan et al. (2008) reported a slump value for NWC of 70mm with a compressive strength of 40 MPa, while the slump varied from 0 to 50mm for LWC using both natural (coarse and fine) and OPBC (coarse and fine) aggregates but with a compressive strength up to 42 MPa. The OPBC after crushing is round in shape which may cause improvement on workability of concrete. For example, the incorporation of the OPBC as coarse aggregate in OPS concrete has significantly improved the workability of the concrete. However, the contribution of fine OPBC in OPS concrete reduced the slump value which was majorly due to the higher water absorption compared to natural mining sand.

2.5.2 Plastic Density

The unit mass or unit weight in air of fresh concrete can be referred to as the plastic density or fresh density of concrete. It can be computed by taking the sum of the masses of all the ingredients of a batch of concrete divided by the volume filled by the concrete. The LWAC using OPBC as both the fine and coarse aggregates was prepared by Abdullahi et al. (2008). They reported that the plastic densities of OPBC concrete were in the range of 1900 to 1970 kg/m^3, depending on the mix proportions and the w/c ratio; as shown in Table 2.5. The air-dry density for OPBC concrete is less than these values yielding a lightweight concrete. Usually, the fresh concrete density of OPBC lightweight concrete is 120 to 460 kg/m^3 more than the saturated density, which might be attributed to the water absorption of OPBC aggregate.

2.5.3 Hardened Density of OPBC Concrete

The compressive strength of concrete depends on the hardened density and is one of the most important variables to consider in the design of concrete structures. The density of structural lightweight aggregate concrete (SLWAC) typically ranges from 1400 to 2000 kg/m^3 compared to that of 2400 kg/m^3 for normal-weight concrete (NWC). According to (Neville, 2008), the density of lightweight aggregate concrete lies between 350 to 1850 kg/m^3, whereas Clarke (2002) reported that the density of LWC varies between 1200 and 2000 kg/m^3. Attempts have been made by many researchers to decrease the density of OPBC concrete without affecting the compressive strength. The density of OPBC concrete depends on various factors, such as, the type of cement, the specific gravity of OPBC, water cement ratio, sand and OPBC content, and water absorption of OPBC. Some researchers studied the use of OPBC as both fine and coarse aggregates and reported that the density of OPBC structural LWC ranged between 1440 and 1850 kg/m^3 and was 23 to 40% lower than the density of NWC (Abdullahi et al., 2008; Kanadasan & Razak, 2014b; Mohammed et al., 2014; Mohammed, Hossain, et al., 2011; Mohammed et al., 2013). Few researchers Mannan & Neglo (2010) and Zakaria (1986) studied the combination of OPBC (as coarse) and natural sand (as fine) aggregates. They stated that the density of OPBC structural LWC lay in the range of 1800 to 2000 kg/m^3. Whereas Ahmad et al. (2007a) used the same materials, OPBC and natural sand as aggregates, but replaced the cement by weight with 10% fly ash. They computed the density range as 1880 to 2030 kg/m^3 and approximately 16 to 22% lower than the density of the NWC. It was observed that by incorporating the 10% fly ash as a binder in OPBC concrete, the density of the LWC was increased in the range of 1 to 4% compared to the mixes without fly ash. A comparative study using a mixture of both ordinary aggregates and LWA (OPBC as fine and coarse) was conducted by Hassan et al. (2008). The range for LWC density varied between 1820 and 2345 kg/m^3 and was 2 to 24% lower than the density of NWC; as shown in Table 2.5. It was reported by Shafigh et al. (2014) that by incorporating OPBC sand in OPS lightweight concrete up to 50% of the weight of normal sand, the density of OPS concrete further reduced from 1990 to 1920 kg/m^3. Further, (Trumble & Santizo, 1993) stated that in the construction of a tower, LWC was considered for use in the floors, whereas NWC was considered for use in the columns. The volume of slabs is usually between 70% and 90% of the total volume of the concrete used in the building (CEB/FIP, 1977). Moreover, strength is not a major consideration in floor slabs; therefore, a large amount of LWC is used to decrease the dead load of the concrete in the floors of multi-storey buildings (Mehta & Monteiro, 2006).

Table 2.5: Selected mix proportions of OPBC concrete and its fresh and mechanical properties

Mix design and Specimens details	Concrete								Aggregates					Tensile Strength (MPa)		Young's Modulus (GPa)	References
	w/c ratio	Mix proportions	Cement content (kg/m^3)	Slump (mm)	Density (kg/m^3)		Strength (MPa)		Size (mm)	Type	Density (kg/m^3)		Water abs: 24h %	Flexural	Splitting		
					Plastic	Hardened	7 days	28 days			Loose	Comp:					
S2 S3 S4 S7 S8 S10 S11 S12 S14	0.32-0.56	OPC+POC(C+F) OPC+POC(C+F) OPC+POC(C+F) OPC+POC(C+F) OPC+POC(C+F) OPC+POC(C+F) OPC+POC(C+F) OPC+POC(C+F) OPC+POC(C+F)	350-600	45-195	1903-1970	1440-1850	13.73 10.20 12.84 19.90 17.37 19.35 15.02 10.21 12.65	22.88 17.00 21.40 33.17 28.95 32.25 25.03 17.02 21.08	<5 5-14	Fine Coarse	1122 793	-	14.29 5.39	-	-	-	(Abdullahi et al., 2008)
A1 A2 A3 A4 A5	0.40-0.46	OPC+POC(C+F) OPC+POC(C+F) OPC+POC(C+F) OPC+POC(C+F) OPC+POC(C+F)	480-520	-	-	1845.6 1835.7 1832.9 1820.5 1818.2	-	42.56 32.08 27.15 26.52 25.50	<5 5-14	Fine Coarse	1119 781	-	26.45 4.35	4.64 4.38 4.01 3.64 3.46	2.72 2.51 2.26 1.90 1.85	26.94 19.35 16.87 12.61 9.73	(Mohammed et al., 2014)
B-1 B-2 B-3 B-4 B-5 B-6	0.60 0.80 0.82 0.60 0.80 0.82	OPC + Sand (F)+POC(C) OPC + Sand (F)+POC(C) OPC + Sand (F)+POC(C) OPC + Sand (F)+POC(C) OPC + Sand (F)+POC(C) OPC + Sand (F)+POC(C)	350-450	100 40 40 40 60 70	-	1868.0 1845.0 1931.0 1946.0 1943.0 1980.0	-	33.00 27.00 32.50 34.00 34.50 35.50	<5 20	Sand Coarse	-	1513 805	0.95 2.65	-	-	-	(Mannan and Neglo, 2010)
S1 S2 S3 S4 S5	0.55 0.55 0.55 0.55 0.55	Control (NWC) OPC+POC (C) + Sand OPC+POC(C+F) OPC & F.A (B), POC(C+F) OPC & F.A (B), POC(C)+Sand	285-510	190 105 85 125 100	-	2486.0 2018.0 1846.0 2026.0 1878.0	40.0 33.3 24.0 37.0 29.1	50.0 41.6 34.0 42.0 36.5	< 4.75 20	Fine Coarse	-	864 -	4.65 1.79	8.02 6.40 5.43 6.43 5.46	4.20 3.25 2.30 3.30 2.30	-	(Ahmad et al., 2007a)
M-1 M-2 M-3	0.20 0.40 0.60	OPC+POC(C+F) OPC+POC(C+F) OPC+POC(C+F)	-	-	-	1837.8 1830.3 1820.1	-	46.8 31.5 20.3	<5 5-14	Fine Coarse	1119 781	-	26.45 4.35	3.46-4.64	2.7 2.3 1.8	26.8 16.9 9.7	(Mohammed et al., 2013)
1:2:3S 1:2:3C 1:2:4S 1:2:4C 1:2:5S 1:2:5C	0.70 0.80 0.90 1.00 0.70 0.70 0.80 0.90 1.00 0.70 0.70 0.70	OPC + Sand (F)+POC (C) OPC + Sand (F)+POC (C) OPC + Sand (F)+POC (C) OPC + Sand (F)+POC (C) OPC + POC(C+F) OPC + Sand (F)+POC (C) OPC + Sand (F)+POC (C) OPC + Sand (F)+POC (C) OPC + Sand (F)+POC (C) OPC + POC(C+F) OPC + Sand (F)+POC (C) OPC + POC(C+F)	-	1.0 8.0 Collapse 15.0 - 2.0 4.0 6.0 10.0 5.0 - -	-	1800 - 2000	20 - - - - 18 - - - 15 6 12	28 - - - - 23 - - - 26 15 21	< 4.75 20	Fine Coarse	-	1075 815	4.65 1.79	5.2 - - - - 5.0 3.8 - - - 3.2 3.0	2.0 - - - - 2.2 1.3 - - - 2.5 1.7	-	(Zakaria, 1986)
POCC-0% POCC-25% POCC-50% POCC-75% POCC-100%	0.50 0.50 0.50 0.50 0.50	OPC + Sand (F)+Gravel(C) OPC + POC(F+C) & (S+G) OPC + POC(F+C) & (S+G) OPC + POC(F+C) & (S+G) OPC + POC(F+C) & (S+G)	450	70 50 45 5 0	-	2345 2214 2083 1952 1821	30.9 32.9 27.3 24.6 22.8	40.3 41.6 35.1 29.7 26.2	< 5 5-14	Fine Coarse	-	1120 840	-	-	-	-	(Hassan et al., 2008)
M M12.5 M25 M37.5 M50	0.38 0.38 0.38 0.38 0.38	OPBC 0%+Sand 100% + OPS OPBC12.5%+Sand87.5%+OPS OPBC 25%+Sand 75% + OPS OPBC37.5%+Sand 62.5%+OPS OPBC 50%+Sand 50% + OPS	500	110 76 76 52 30	1980 1976 1966 1917 1881	1996 1989 1993 1948 1919	34 33 34 30 28	37 36 37 33 32	< 4.75 9.5	Fine (OPBC) & (Sand) Coarse (OPS)	1025 1525 561	1137 1657 626	-	4.18 4.15 3.84 3.73 3.22	2.64 2.57 2.49 2.41 2.20	13.8 13.0 12.6 12.4 8.6	(Shafigh et al., 2014)

2.6 Mechanical Properties of OPBC Concrete

2.6.1 Compressive Strength

The compressive strength of structural concrete is the most appropriate property for any innovative material used in concrete technology. It affects all the other mechanical properties of concrete such as flexural strength, splitting tensile strength and modulus of elasticity. According to ACI, the cylindrical compressive strength of LWC at 28 days should not be less than 17 MPa (Neville & Brooks, 2008).

Researchers have studied different LWAs, mix design and curing conditions to achieve varying grades of strength. Some of them used OPBC as both fine and coarse aggregates to develop a LWAC known as oil-palm-boiler clinker concrete (OPBCC). Abdullahi et al. (2008) have studied properties of LWC by using of OPBC as both coarse and fine aggregates in concrete mixture. Firstly, they selected a w/c ratio in the range of 0.32-0.56 and succeeded in developing a compressive strength range of 17 to 33 MPa. Further, they continued their research to investigate the flexural strength of the same concrete, but this time they selected the w/c ratio in the range of 0.40 and 0.46, and achieved a higher compressive strength up to 47 MPa (Mohammed et al., 2014; Mohammed et al., 2013); as shown in Table 2.5. Ahmad et al. (2007a) comparatively studied NWC by using the DoE method and LWC using the FIP design method. They designed five different mix proportions, one for NWC and four for LWC at the same w/c ratio of 0.5. The compressive strength achieved for NWC was about 50 MPa, whereas for LWC, in the first proportion, the materials preferred were binder (cement), OPBC (C) and natural sand as aggregates at the same w/c ratio. This proportion gave an average compressive strength of 42 MPa. It was approximately 16 % lower than the compressive strength of NWC. In the second mix proportion, the preferred materials were the same, however, instead of natural sand, fine OPBC was used. The reduction observed in the compressive strength was up to 32%. Then, for the other two mix proportions, the materials preferred were the same as in the first and second mixes; however, fly ash was added with a binder up to 10% of OPC content. Nevertheless, the compressive strength was not affected by using the fly ash, the compressive strength was 16 to 28% lower than the compressive strength of NWC.

Several researchers studied the use of OPBC as a coarse aggregate for developing a structural LWAC. Zakaria (1986) studied the use of OPBC as a coarse aggregate and natural sand as a fine aggregate. The range selected for the w/c ratio varied between 0.70 and 1.0. Due to the high w/c ratio, he achieved a 28-day compressive strength ranging between 15 and 28 MPa. Later, Mannan and Neglo (2010) studied the same aggregates (OPBC and natural sand) to develop a LWC, but they selected a w/c range between 0.60 and 0.82. Due to the proper selection of the w/c ratio, they achieved better compressive strength results varying between 27 and 36 MPa. The behaviour of LWC by using OPS (coarse), normal and OPBC sands with a constant w/c ratio of 0.38 was studied by Shafigh et al. (2014). The first matrix was considered as the control with cement, OPS and natural sand; whereas, for the other proportions, the natural sand was replaced by OPBC sand at 12.5, 25, 37.5 and 50%. The 28-day compressive strength achieved ranged from 30 to 38 MPa, as shown in Figure 2.8.

Some researchers conducted a comparative study of the behaviour using a mixture of normal weight aggregate (NWA) and OPBC as the coarse and fine aggregates to fabricate LWAC. Hassan et al. (2008) fabricated five different mix proportions by using the ACI design method with the same w/c ratio of 0.50. The first matrix was considered as the control mix of NWC. In the next four matrixes, they replaced the NWA with OPBC aggregate with 25% increments. The compressive strength for the mixes containing 0, 25%, 50%, 75% and 100% OPBC were 40, 42, 35, 30 and 26 MPa, respectively. The 25% replacement of NWA with OPBC was able to give even better compressive strength than

NWC. Later, Kanadasan and Razak (2014a) studied the same materials and mix pattern with different w/c ratios ranging from 0.42 to 0.54. Due to the lower w/c ratio, they achieved even better results for the compressive strength of concrete containing OPBC varying from 35 to 50 MPa compared to control OPS concrete. Test results of a study by Aslam et al. (2015) showed that in OPS lightweight concrete if OPS is substituted with 40% OPBC, the new concrete with this blended coarse lightweight aggregate has higher compressive strength compared to OPS concrete. The main problem of OPS concrete is due to the smooth surface texture of the shell for both concave and convex faces which resulted weak bond in concrete. However, the incorporation of the rough textured OPBC aggregate in OPS concrete has a significant effect on the compressive strength of the concrete.

Figure 2.8 shows the relationship between the water to cement ratio with compressive strength of OPBC concretes at 28 days age. As expected, by increasing the water to cement ratio the compressive strength of OPBC concrete containing coarse or coarse and fine aggregates reduces. This relationship shows that at the same water to cement ratio, concrete containing coarse OPBC has higher compressive strength than the concrete containing coarse and fine OPBC aggregates. This difference is more significant at higher water to cement ratios.

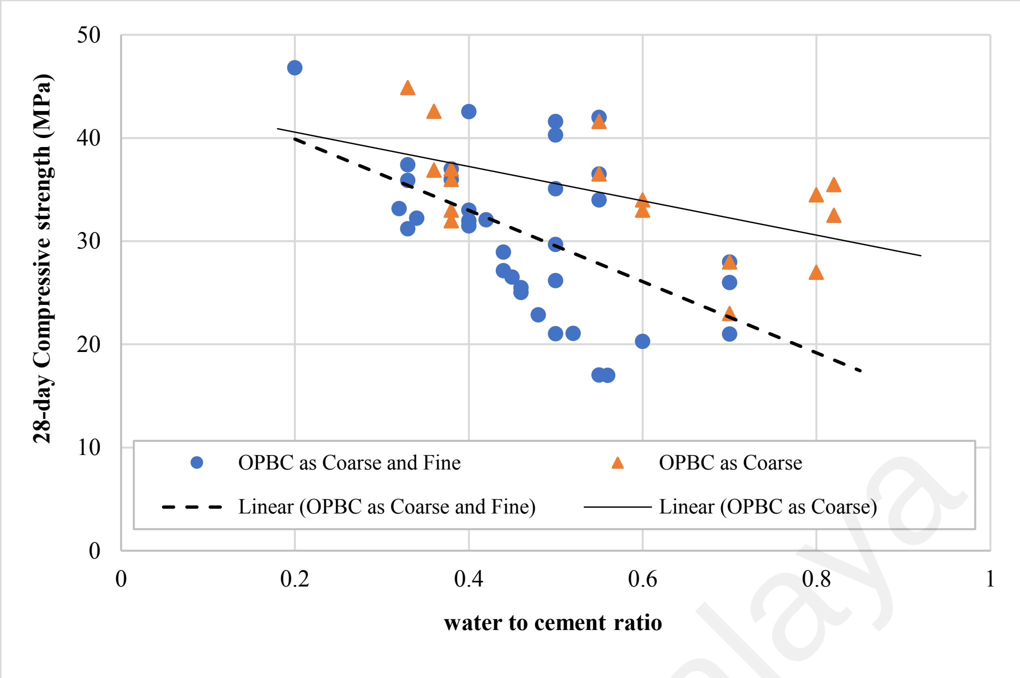


Figure 2.8: Compressive strengths of OPBC concrete at different w/c ratios
Source: (Abdullahi et al., 2008; Aslam et al., 2015; Hassan et al., 2008; Kanadasan & Razak, 2014b; Mannan & Neglo, 2010; Mohammed et al., 2014; Mohammed et al., 2013; Shafigh, Mahmud, et al., 2014; Zakaria, 1986)

2.6.2 Modulus of Rupture

The modulus of rupture (MOR) for OPBC concrete reported by various researchers as can be seen in Table 2.5. Mohammed et al. (2011a) and Mohammed et al. (2014) conducted an experimental investigation using OPBC (as fine and coarse) aggregates. Due to the porous nature of OPBC aggregate, they achieved MOR values in the range of 3.5 to 4.6 MPa and approximately 10% of the compressive strength of OPBC concrete. Zakaria (1986) performed an experimental investigation using normal sand and OPBC as the coarse aggregates. He reported that the MOR value for this concrete ranged between 3.0 and 5.0 MPa and was approximately 20% of the compressive strength of concrete. Ahmad et al. (2007a) reported different mix designs using OPBC and normal aggregates. The two mixes were prepared using OPBC as the coarse aggregate and natural sand as the fine aggregate. It was observed that replacing OPC (binder) with 10% fly ash reduced the flexural strength of the LWC up to 15% of the LWC without fly ash. The flexural strength of the mixes ranged between 5.5 and 6.4 MPa. However, in the other two mixes, which contained OPBC as the coarse and fine aggregates, it was reported that replacing OPC with 10% fly ash could enhance the modulus of rupture up to 15% of the mix without fly ash. The MOR values for these mixes were found to be in the range of 5.4 to 6.4 MPa, and these values were almost 15% of the compressive strength of the LWC. The MOR value is affected from the mix design proportions. Mohammed et al. (2014) and Ahmad et al. (2007a) used nearly similar w/c ratios, but with a higher binder (fly ash) content, Ahmad et al. (2007a) reported an increment in the MOR of the lightweight concrete. A similar trend was reported by Mahmud et al. (2009), albeit with the use of cementitious materials. It was further reported that the MOR increased when the amount of cement content increased, and decreased when the w/c ratio increased (Mohammed, Hossain, et al., 2011; Mohammed et al., 2013).

The relationship between the MOR and compressive strength was further investigated using the different equations suggested by a number of standards and researchers. The fabrication of the LWAC was examined with OPBC as coarse and fine aggregates, as well as with natural sand and OPS. The binders can be used normally or mixed with fly ash for the fabrication of LWAC. There are several proposed equations to predict the MOR from the compressive strength (CEB/FIP, 1977; Shafigh, Jumaat, Mahmud, & Norjidah, 2012). However, not all of them can be used for OPBC concrete. Using the existing data, Equation 2.1 gives a good estimation of the MOR for OPBC concrete from the compressive strength with a R^2 of 70 to 90%. However, as a specific equation, Equation 2.2 could be used. This equation (CEB/FIP, 1977) was previously suggested for predicting the MOR of LWAC with a cube compressive strength in the range of 20 to 60 MPa. The Equation 2.2 gives about 85% reliability between the experimental and predicted values.

$$f_r = \mathrm{K}\sqrt{f_{cu}} \tag{2.1}$$

$$f_r = 0.46\sqrt[3]{f_{cu}^2} \qquad (2.2)$$

where, f_r and f_{cu} are the 28-day flexural and compressive strengths in MPa, and "K" is the constant parameter ranging between 0.73 and 0.76.

A comparison between the existing experimental data for OPBC concrete with the predicted modulus of rapture from the equations proposed by (ACI363, 1992); (Lo, Cui, & Li, 2004), IS (Short, 1978), and the proposed equation for high strength lightweight aggregate concrete (HSLWAC) (CEB/FIP, 1977) shows that the equations suggested by ACI and IS gives almost similar results (an error below 10%) to the experimental values. While, the equation proposed for HSLWAC gives very conservative estimation for all OPBC concretes.

2.6.3 Splitting Tensile Strength

The compressive strength is the property of concrete normally considered in a structural design, however, for some purposes, such as the design of airfield slabs and highways, resistance to cracking, the shear strength and the tensile strength are of interest (Neville, 2008). Mohammed et al. (2013; 2014) reported that when using OPBC as fine and coarse aggregates, the splitting tensile strength value ranged between 1.8 and 2.7 MPa with the w/c ratio varying from 0.20 to 0.60, and was around 6 to 10% of the compressive strength. They further reported that the splitting tensile strength increased when the cement content increased, and decreased when the w/c ratio increased. Ahmad et al. (2007a) fabricated LWC using OPBC aggregate and binders (OPC and fly ash); the observed splitting tensile strength varied from 2.30 to 3.30 MPa, and was 6 to 8% of the compressive strength. This range of splitting tensile strength was about 16 to 32% less than the control normal weight concrete. They reported that the splitting tensile strength of the concrete was dependent on the splitting tensile strength of OPBC aggregate.

Zakaria (1986), who studied the mixture of natural sand and OPBC (as coarse), found that the splitting tensile strength of concrete varied from 1.3 to 2.5 MPa with a very high w/c ratio of 0.70 to 1.0, and was about 6 to 16% of the compressive strength. As reported by Al-Khaiat & Haque (1998) that the splitting tensile strength of structural LWC was less than the tensile strength of similar strength grade NWC. Furthermore, Shafigh et al. (2014) performed a detailed study using OPS (as coarse), normal and OPBC sands. The splitting tensile strength ranged between 2.2 and 2.6 MPa. They reported that the rate of compressive strength gained with time was more significant than the rate of the splitting tensile strength.

The relationship between the splitting tensile strength and the compressive strength was further investigated using the different equations suggested by a number of standards and researchers. The fabrication of the LWAC was examined when OPBC was used as coarse and fine aggregates, or with the natural sand and OPS. Equation 2.3 shows a general relationship between the splitting tensile strength and compressive strength (ACI363, 1992; Slate, Nilson, & Martinez, 1986). In this form of proposed equation, if the constant K varied between 0.46 and 0.53, the splitting tensile strength of OPBC concrete could be predicted with a 77 to 91% reliability. The equation suggested by (Shetty, 2005) for the splitting tensile strength of NWC was also implemented for OPBC concrete. It was observed that this (Equation 2.4) gave more reliable results of approximately 90% of the experimental results for the splitting tensile strength of OPBC lightweight concrete. An equation for the splitting tensile strength of the LWC with expanded clay and shale with a cube compressive strength ranging between 20 and 60 MPa, was also investigated (Shafigh et al., 2010). It also gave very high reliability ranges between 70 and 97% for OPBC concrete, as shown in Equation 2.5.

$$f_t = K\sqrt{f_{cu}} \qquad 0.46 < K < 0.53 \qquad (2.3)$$

$$f_t = 1.05 + 0.005f_{cu} \tag{2.4}$$

$$f_t = 0.27\,(f_{cu})^{0.63} \tag{2.5}$$

where, f_t is splitting tensile strength and f_{cu} is the 28-day compressive strength in MPa.

The experimental data for OPBC concrete was also compared with the predicted splitting tensile strength from the equations proposed by ACI 363 (1992), CEB/FIP, (1977), and NWC (Shetty, 2005). It was observed from the results that the equations suggested by ACI, CEB-FIP and NWC showed closer estimations (an error in the range of 5-15%) to the experimental values. If it be assumed that a prediction error of up to 10% is acceptable then all these equations can be used for OPBC concretes.

2.6.4 Modulus of Elasticity

The Young's modulus of elasticity or E-value of the concrete is one of the most important parameters in the design of structural members. The modulus of elasticity of a material is a measure of its stiffness, and, for most materials, remains constant over a range of stresses. The E-value of the concrete depends upon the moduli of elasticity of its constituents and their quantities by volume in the concrete; its value is reduced when NWAs were replaced by LWAs (Neville, 2008). The modulus of elasticity of LWAs was less than NWAs and was even smaller than mortar, ranging mostly from 5 to 28 GPa (CEB/FIP, 1977). Therefore, it is projected that by enhancing the volume of LWA and reducing the volume of the normal aggregate, there is a reduction in the modulus of elasticity of concrete. Generally, the modulus of elasticity of structural LWC ranges between 10 and 24 GPa, while, in NWC, it ranges from 14 to 41 GPa (Shafigh et al., 2014). Only a few researchers have focused on this important property in their study. Mohammed et al. (2013; 2014) used OPBC as both fine and coarse aggregates with different mix proportions and w/c ratios. The E-value varies from 9.7 to 27 GPa. Shafigh et al. (2014) considered different materials, such as OPS as lightweight coarse aggregate, normal and OPBC sands as fine aggregates with different mix proportions, and attained a varying range of modulus of elasticity between 8.6 and 13.8 GPa. They reported that the modulus of elasticity of OPS concrete containing 37.5% (percentage of total sand) of OPBC sand is in the normal range for structural LWA concrete.

The relationship between the modulus of elasticity (E) and compressive strength (f_{cu}) was further investigated using the different equations suggested by some researchers. The fabrication of the LWAC was examined when OPBC was used as both coarse and fine aggregates. The general equation (2.6) proposed by ACI318R (2005) for the modulus of elasticity of NWC was investigated for OPBC lightweight concrete. It was observed that this equation gave an 80% reliability between the experimental and theoretical results. The equation suggested by BS-8110 (1986) for the modulus of elasticity of the NWC was also implemented for OPBC concrete. It was observed that this (Equation 2.7) gave good reliability for OPBC lightweight concrete.

$$E = 0.043\ \omega^{1.5}\ (f_{cy})^{0.5} \qquad (2.6)$$

$$E = 0.0017\ \omega^{2}\ (f_{cu})^{0.33} \qquad (2.7)$$

where, E is the expected modulus of elasticity (MPa), ω is the air-dry density (kg/m^3); f_{cu} and f_{cy} are the cubical and cylindrical compressive strengths (MPa).

The experimental data for OPBC concrete was also compared with the equations proposed by ACI 318 (2005), CEB/FIP (1977) and BS-8110 (1986). The results showed that for OPBC concretes, the models suggested by ACI, BS-8110 and NWC gives closer results (an error up to 13%) to the experimental values. While, the equation of CEB-FIP gives very conservative results for all OPBC concretes.

2.7 Structural Behaviour of OPBC Concrete

2.7.1 Flexural Behaviour

The flexural behaviour of reinforced concrete (RC) beams produced by OPBC (as coarse and fine aggregates) was reported by Mohammed et al. (2014). Eight beams, four each, singly and doubly reinforced, and were tested with different reinforcement ratios varying between 0.34 and 2.21%. It was observed that all OPBC concrete beams showed the typical flexural failure mode, whereas, the flexural cracks originated first in the constant moment zone. The beams were designed as under-reinforced; therefore, the failure was initiated by yielding of the tensile steel reinforcement before the compression failure of the concrete. Generally, similar to the NWC beams, the ductility ratio of the reinforced OPBC concrete beams reduced as the steel reinforcement ratio increased. Mohammed et al. (2014) further suggested that due to the increasing load, the effect of flexure and shear combination created new flexural cracks in the shear span and curved diagonally. They further computed the ultimate moment (M_{ult}) and the theoretical design moment (recommended by BS-8110), and claimed that OPBC concrete beams with reinforcement ratios of 2.23 or less the ultimate moment (M_{ult}) obtained experimentally was approximately 1 to 7% higher than the predicted value. Further, they recommended that the BS-8110 based design equations be used for the prediction of the flexural capacity of OPBC concrete beams with reinforcement ratios up to 2.23% because most of the ductility index observed in OPBC concrete beams was 2.71. From this result, they reported that the reinforced OPBC concrete beams exhibited lower ductility behaviour with the increase in the reinforcement ratio. In addition, the deflection under the design service loads for singly reinforced OPBC concrete beams with a reinforcement ratio less than 0.5% was within the allowable limit provided by BS-8110. Therefore, it was suggested that for beams with a higher reinforcement ratio, beam depths should be increased. Six under-reinforced beams with varying reinforcement ratios (0.52% to

3.90%) were designed by Teo, Mannan, & Kurian (2006). They reported that the ultimate moments predicted using BS-8110 provided a conservative estimate for OPS concrete beams up to a reinforcement ratio of 3.14%, with experimental ultimate moments of approximately 4% to 35% higher compared to the predicted moments. For the beam with a 3.90% reinforcement ratio, BS-8110 underestimated the ultimate moment capacity by about 6%. Further, it was observed that in OPS concrete beams with reinforcement ratios up to 2.01%, the ductility ratio was more than 3, which showed relatively good ductility. It was also reported that a higher tension reinforcement ratio resulted in lower ductile behaviour. Therefore, it could be seen that for both types of lightweight aggregate originating from the palm oil industry, the steel reinforcement ratio of RC beams should be limited to a certain amount. In general, the reinforcement ratio of RC beams for normal weight concrete beams varied from 0.3 to 8 % of the gross cross-sectional area of the concrete (Narayanan, Wilson, & Milne, 2000). While for LWC, the British and American codes do not differentiate between the lightweight and normal weight concrete under flexure; however, the draft European code considers lightly stressed and highly stressed members, based on the level of stress in the reinforcement. As a guide, it says that members with less than 0.5% of tension reinforcement can be taken as being lightly stressed, as is normally the case for slabs. Highly stressed members are assumed to have 1.5% of reinforcement or more (Clarke, 2002).

Mohammed et al. (2014) also reported that the crack widths for OPBC concrete beams at service loads ranged from 0.24 mm to 0.30 mm while it is 0.22 mm to 0.27 mm for OPS concrete beams (Teo et al., 2006). The researchers reported that the crack widths for OPS and OPBC concrete beams were found in the allowable limit as recommended by ACI 318 and BS-8110 for durability requirements. ACI 318 permits the crack widths up to 0.41 mm, whereas this limitation for BS-8110 is 0.3 mm (Mohammed et al., 2014). Further, Hussein, Mustapha, Muda, & Budde (2012) carried out modelling works on reinforced OPBC beams with web openings using response surface methodology (RSM). The authors revealed that the use of RSM was capable of predicting the ultimate load of reinforced POC beams with web openings with an accuracy of 99.27%.

Mohammed et al. (2011a) used OPBC aggregate to fabricate composite floor slab with profiled steel sheeting and compared its flexural performance with the corresponding conventional NWC slab with both long and short shear spans. In the study, it was found that both types of slab failed in a similar manner, which was shear-bond failure. Nevertheless, all the slabs were considered to exhibit ductile behaviour. The deflection of the OPBC slab was found to be higher compared to conventional NWC slab and the authors reasoned this to the lower modulus of elasticity of OPBC. Despite this, the authors concluded that the behaviour of the OPBC slab was satisfactory and could be used for the construction of composite slabs. Since one of the main advantages of using composite deck slab is being a lightweight structure building material, the use of OPBC for such purpose could further enhance its application since the OPBC slab was found to have about 18% lower self-weight compared to NWC slab in this study.

2.7.2 Shear Behaviour

In terms of structural behaviour, the shear strength of concrete is considered as the most controversial topic (Kong & Evans, 1998). Mohammed et al. (2013) fabricated seven reinforced OPBC concrete beams to investigate the shear behaviour. All the beams were designed as under-reinforced and the compressive strength of selected OPBC concrete was about 20, 32 and 40 MPa with an air-dry density of 1820, 1830 and 1838 kg/m^3. They considered three main parameters to investigate the shear behaviour of OPBC concrete beams: the shear span to effective depth ratio (a/d), tension reinforcement ratio (ρ) and compressive strength of OPBC concrete. They reported that the structural behaviour and failure mode of the reinforced OPBC concrete beams under shear were similar to that of the conventional RC beam. The experimental results were also compared with the shear strength design equation proposed by the Canadian Standard Association (CSA). For the beams having a shear span to effective depth (a/d) ratio less than 2, the bending cracks did not develop, but, suddenly, the shear cracks developed at 45° and ran through the compression zone and produced collapse. However, for the beams having 2<a/d<6, the initial bending appeared and became inclined early in the loading stage, and, at collapse, horizontal cracks were produced and ran parallel to the tensile reinforcement. These horizontal cracks decreased the shear resistance of the member by destroying the dowel force and reduced the bond stresses between OPBC concrete and steel. Further, it was noted that the shear-cracking mode of reinforced OPBC concrete beams was very similar to that of conventional reinforced concrete beams without shear reinforcement.

The theoretical shear resistance of reinforced OPBC concrete beams (V_c) was computed using different equations, as given in ACI-318 (2002) (Eq: 2.8) and CSA (Eq: 2.9), and as given below (Mohammed et al., 2013):

$$V_c = \frac{\lambda\sqrt{f_c'}}{6} b_w d \tag{2.8}$$

$$V_c = \beta \varphi_c \lambda \sqrt{f_c'}\, b_w d \tag{2.9}$$

where f_c' is the standard cylinder compressive strength, b_w is the width of the beam, d is the effective depth of the beam, β is the factor accounting for the shear resistance of the cracked concrete = 0.18, φ_c is the resistance factor of concrete having the value 0.65 and λ is the factor for low density concrete = 0.75.

Mohammed et al. (2013) generally found four different types of failure: (1) if a/d > 6; the failure mode was similar to that of pure bending, (2) when 2<a/d<6; the failure mode was in shear due to the anchorage failure between the steel and concrete, (3) if a/d < 2;

the failure mode was in shear due to compression zone failure, and finally, (4) if a/d = 0; the punching shear failure was occurred. It should be noted that the shear strength has been expressed as the failure moment obtained by multiplying it with the shear span (a). The failure of the beam governed by a/d ratio could be classified in four groups. When the a/d ratio was less than 1, the beams failed in splitting or experienced compression failure. This normally appeared as an inclined thrust between the load and the support point. However, when the a/d ratio ranged from 1 to 2.5, it forced the beams to fail in shear compression. Diagonal tension failure occurred when the a/d ratio was in the range of 2.5 to 6. Flexural failure occurred when the a/d ratio is greater than 6. This failure was initiated due to the yielding of the tension steel reinforcement (Sinha, 2002).

Clarke (2002) studied the available data and carried out additional tests on beams, both with and without shear reinforcement by using two artificial LWAs as well as pumice. The resulting strengths were 30 and 50 MPa at densities of 1700 and 2000 kg/m^3. It was concluded that without shear reinforcement, a 0.9 reduction factor would be more appropriate. For beams with links, no reduction factor was necessary. Furthermore, it was reported that by replacing 0.8 with 0.9 led to an effective reduction factor of 0.94 with nominal shear reinforcement. Lightweight aggregate beams with shear reinforcement will carry 90% capacity compared to normal weight beams. It was also reported that the a/d ratios for lightweight aggregate concrete was 0.85 times that for normal weight concrete (Clarke, 2002).

2.7.3 Behaviour of OPBC Concrete under Prestressing

LWA concrete has been used in place of NWC in building applications to reduce the dead load and dimensions of the members. Recently, researchers and bridge engineers have wanted to implement the use of normal strength LWC and high strength LWC for similar reasons. It was reported by Bender (1980) found that the LWC was very economical, especially for long span structures. Roslli and Mohamed (2002) studied and compared the performance of prestressed OPBC lightweight concrete beams with normal prestressed concrete beams under two-point static loading. They designed four pre-tensioned prestressed concrete beam specimens without web reinforcement; two beams were cast using OPBC (as coarse and fine aggregates) and another two beams using normal weight aggregates. Both types of concrete were designed to achieve the targeted strength of 40 MPa, which is the minimum required strength for pre-tensioned prestressed structures (BS-8110, 1986). The concrete batches were designed for a workable and cohesive mixture. The physical and mechanical properties of OPBC and normal concretes used for pre-tensioned beams are mentioned in Table 2.6.

Table 2.6: Physical and mechanical properties of the pre-tensioned concretes

Properties	OPBC lightweight concrete	Normal weight concrete
Fresh unit weight (kg/m^3)	1950	2370
Cubical compressive strength (MPa)	46.0	44.0
Modulus of rupture (MPa)	6.1	6.4
Tensile strength (MPa)	3.2	3.3
Modulus of Elasticity (GPa)	22.0	30.0

Source: (Roslli & Mohamed, 2002)

Roslli and Mohamed (2002) also reported the cracking behaviour in prestressed OPBC concrete beams. The cracks appeared in the early stages because of the lower cracking load and low tensile strength as compared to NWC. The crack lines in OPBC concrete propagated smoothly and almost linear during load increment. This was due to the softer and weaker properties of OPBC concrete, while for the NWC with high strength aggregates the crack lines propagated roughly. The difference in the load carrying capacity for prestressed lightweight concrete beams was only 6.4%. The deflection at maximum load for prestressed OPBC concrete beams was approximately 25% higher than for the prestressed normal weight concrete beams. This is because materials with a lower modulus of elasticity deform more than stiffer materials (Shuaib & Barker, 1991). It was also found that OPBC concrete beams tend to have higher deflection of about 32% at lower failure load compared to the NWC specimens. The failure mode of both NWC and OPBC concrete beams showed a very similar result. All beam specimens failed in shear with almost similar maximum strength.

2.8 Conclusions

The application of oil-palm-boiler clinker (OPBC) as lightweight aggregate (LWA) to produce OPBC concrete was reviewed through existing literature. The physical, mechanical, chemical, durability, functional and structural behaviour of OPBC concrete were discussed and compared with other lightweight aggregates. The behaviour of OPBC concrete was compared also with conventional concrete. Based on the literature review, the following conclusions can be drawn:

1. OPBC is in lump of irregular shapes with rough and spiky broken edges and sizes varying from 150 to 225 mm, whitish grey in colour and has the appearance of a porous stone. OPBC can be termed as lightweight aggregate with a specific gravity of 1.70-2.20. OPBC aggregate is 15-30% lighter than NWAs and 47-53% heavier than artificial LWAs.
2. The OPBC aggregate is a porous material and it will absorb high amount of water compared to the normal weight aggregates. The 24-h water absorption for fine and coarse OPBC aggregates ranges from 4.7 to 26.5% and 1.8-5.4%, respectively.
3. The Los Angeles abrasion and crushing values of OPBC aggregates are much lower than agricultural solid wastes such as oil palm shell and coconut shell. However, the values of these properties for OPBC aggregates are in suitable range suggested by the codes, and show that it can be used in all types of structures. The

main reason for lower abrasion and crushing values is that the shells are similar to wood and are relatively ductile materials while an OPBC is a brittle material.

4. OPBC is stated to have very low workability as seen from slump values due to rough, spiky and angular edges. Though, the super-plasticizer can be used to solve the low workability problem.
5. The hardened density of OPBC concrete was found in the range of 1440-2212 kg/m^3.
6. When OPBC was used as coarse aggregate with normal sand, the 28-day compressive strength was found in the range of 15-42 MPa with the density of 1800-2000 kg/m^3. However, concretes containing both coarse and fine OPBC aggregates may have a compressive strength in the range of 17-47 MPa with a density of 1440-1850 kg/m^3.
7. The self-compacting concrete (SCC) can be successfully produced using OPBC aggregates and particle packing method. The results indicated that the mixture design could be employed not only for OPBC but also for a variety of combinations of aggregates. It not only helps to conserve the natural resources but also promotes sustainability in preserving the environment.
8. The replacement of natural mining sand with OPBC sand in OPS concrete reduces the compressive strength of the concrete. However, the substitution level of OPBC sand was optimized up to 25% in OPS concrete.
9. The OPBC concrete showed the modulus of rupture in the range of 10-20% of the compressive strength, whereas this range is about 6-10% for splitting tensile strength. However, the modulus of elasticity of this lightweight concrete was found in the range of 8.5-27.0 GPa.
10. The reinforced OPBC concrete beams show the typical flexural failure mode, and displayed higher ductile behaviour compared to the NWC beams. The moment

curvatures of the beams of OPBC concrete also followed the same trend as that of NWC with a crack widths at service loads in the range between 0.24 mm and 0.30 mm.

11. The structural behaviour and the failure mode of OPBC reinforced concrete beam under shear is similar to that of conventional reinforced concrete beam, and the aggregate interlock property increases the shear strength in OPBC concrete.
12. The use of response surface methodology (RSM) on reinforced OPBC beams is capable of predicting the ultimate load of reinforced OPBC beams with web openings with an accuracy of 99.27%.
13. The OPBC aggregate can contribute to good bonding strength with the mortar matrix due to its surface texture and rough plane faces.
14. The deflection of the prestressed OPBC concrete beams was slightly greater than the NWC beams. This was expected due to lower elastic modulus. The cracking behaviour of prestressed OPBC lightweight concrete beams is almost similar to that of prestressed NWC beams.

CHAPTER 3: BENEFITS OF USING BLENDED WASTE COARSE LIGHTWEIGHT AGGREGATES IN STRUCTURAL LIGHTWEIGHT AGGREGATE CONCRETE

3.1 Introduction

Concrete is the most widely used construction material in civil engineering structures. It has an excellent resistance to water and can be formed into a variety of shapes and sizes (Shafigh et al., 2010). Nowadays, the concrete industry consumes enormous amounts of natural resources and raw materials for production (Shafigh et al., 2013). Because of the huge amount of concrete produced daily, even a small reduction in the use of raw materials in concrete mixtures will result in considerable benefits to the environment (Altwair & Kabir, 2010). The best way to achieve sustainability in the concrete industry is to utilize by-products and waste materials (Mannan & Neglo, 2010).

The use of lightweight concrete (LWC) in a structure reduces its overall dead load which can be considerable (Bremner & Eng, 2001). Structural lightweight concrete is usually made with lightweight aggregate (LWA). In most cases, the LWA used in lightweight aggregate concrete (LWAC) is coarse. One LWA that is abundantly available in most tropical countries is oil palm shell (OPS), a solid waste from the palm oil industry. The density of OPS is within the range of most typical lightweight aggregates, with a specific gravity in the range of 1.1-1.4 (Shafigh, Jumaat, Mahmud, et al., 2012). However, reports (Alengaram et al., 2013; Teo et al., 2006) have shown that it has high water absorption in the range of 14-33%.

In the last two decades, OPS have been used as an LWA for producing structural LWAC with a density 20-25% lower than NWC (Shafigh, Jumaat, Mahmud, et al., 2012; Shafigh et al., 2010). Ali, Abdullah, Salam, and Rahim (1984) and Salam, Ali, and Abdullah (1987) introduced the use of OPS as an LWA, achieving a compressive strength of 20 MPa with a water to cement ratio of 0.4. Okafor (1988) reported a compressive strength of 25-35 MPa for OPS concrete, which is in the range of typical compressive strength for structural lightweight concrete (Kosmatka et al., 2002). Mannan and Ganapathy (2001b) reported that depending on the curing condition, the 28-day compressive strength of OPS concrete ranged between 20 and 24 MPa with a water to cement ratio of 0.41. Later, they stated that OPS aggregate could be treated by using a 20% polyvinyl alcohol solution, which significantly reduces water absorption and provides a better interlock with cement paste. They reported that lightweight concrete containing treated OPS has a 40% higher compressive strength than the control OPS concrete (Mannan, Alexander, Ganapathy, & Teo, 2006). Yew, Mahmud, Ang, and Yew (2014) also examined the heating method for treatment of OPS coarse aggregate. They reported that the advance in heat treatment methods has no significant effect on the compressive strength. However, it enhances workability by about 20% and slightly reduces the density of the OPS concrete. They achieved a 28-day compressive strength of 49 MPa for OPS concrete using treated OPS. Alengaram, Mahmud, and Jumaat (2011) achieved the highest compressive strength, reaching about 37 MPa by using silica fume and class F fly ash. Recently, high strength OPS lightweight concrete was successfully produced by (Shafigh, Jumaat, & Mahmud, 2011; Shafigh, Jumaat, Mahmud, & Alengaram, 2011; Shafigh, Mahmud, & Jumaat, 2011) with a compressive strength in the range of 42-53 MPa. The key characteristic of mix proportions of a high strength OPS concrete is the use of crushed OPS, smaller sizes of coarse OPS, low water to cement ratio and the use of limestone powder in the concrete mixture as filler (Shafigh, Jumaat, & Mahmud, 2011). Recently, Mo et al. (2016) investigated the durability properties of a sustainable concrete by using OPS as coarse and manufactured sand as fine aggregates. They reported that the use of GGBFS as partial cement replacement in the OPS concrete increased the compressive strength gain compared to OPS concrete without any GGBFS over the curing period of 1 year.

A report by Teo, Mannan, and Kurian (2009) revealed that when normal coarse aggregate is substituted with OPS, mechanical properties such as compressive, splitting tensile and flexural strengths are reduced by about 48%, 62% and 42%, respectively. However, a maximum reduction of up to 73% was observed for the modulus of elasticity. The modulus of elasticity of OPS concrete is in the normal range of 5.5-11 GPa, which is considered a low value in this property (Alengaram, Mahmud, et al., 2011; Mannan & Ganapathy, 2002). However, Shafigh, Mahmud, and Jumaat (2012) reported that for high strength OPS concrete, it could increase up to 18 GPa. It should be noted that, generally, the modulus of elasticity of structural lightweight concrete ranges between 10 and 24 GPa (CEB/FIP, 1977). A low value causes excessive deformation in structural elements such as slabs and beams. The durability properties of different grades of OPS concrete have also been studied by several researchers (Haque, Al-Khaiat, & Kayali, 2004; Mannan et al., 2006; Teo, Mannan, Kurian, & Ganapathy, 2007), and their reports have shown that this concrete can be considered a durable.

Although previous studies have shown that OPS concrete has satisfactory mechanical and durability properties, it does have some drawbacks that need to be addressed before it can be used in real structures. One of the main drawbacks is sensitivity of compressive strength in poor curing conditions. Compared to normal aggregate concrete, OPS show a significant reduction in strength when not properly cured (Mannan & Ganapathy, 2002; Shafigh, Jumaat, & Mahmud, 2011). Mannan and Ganapathy (2002) reported that OPS concrete subjected to 7-day moist curing showed 17% lower compressive strength than OPS concrete under full water curing. High strength OPS concrete is also sensitive to poor curing. A minimum period of 7 days of moist curing is recommended for this type of concrete (Shafigh, Jumaat, Mahmud, et al., 2012). The sensitivity of compressive strength of OPS concrete increases when the mixture contains a high cement dosage, low water to cement ratio or high OPS content (Shafigh, Jumaat, & Mahmud, 2011). Islam et al. (2016) investigated the fresh and mechanical properties of sustainable OPS lightweight by using agro-solid waste materials. The OPS was used as coarse aggregate while ground POFA was used at partial cement replacement levels of up to 25%. They reported that OPS concrete specimens subjected to continuous moist curing showed higher compressive strength compared to partially cured and air dried specimens. Therefore, they recommended that the incorporation of POFA could reduce the sensitivity of OPS concrete towards poor curing. High cement content and low water to cement ratio are needed to produce a structural grade of LWAC with satisfactory durability properties, particularly when high strength is required. Therefore, changing the volume of OPS content may reduce sensitivity.

The aim of this study was to investigate the possibility of reducing compressive strength sensitivity of OPS concrete by reducing OPS aggregate volume. For this purpose, OPS was partially substituted with oil-palm-boiler clinker (OPBC). OPBC is a by-product of the burning of solid waste in the boiler combustion process in palm oil mills. It is like a porous stone, grey in colour, flaky and irregular in shape (Ahmad et al., 2007a, 2007b). Previous studies (Ahmad et al., 2008; Chan & Robani, 2008; Zakaria, 1986) have shown that OPBC can be used as lightweight aggregate in concrete. The density and the 28-day compressive strength of OPBC concrete fulfil the requirements of structural LWAC (Mohammed et al., 2014). OPBC was chosen as a partial replacement as it is a waste without any current application, as well as being a lightweight aggregate. Because there are no reports pertaining to the compressive strength sensitivity of OPBC concrete in poor curing conditions. This study used two types of waste coarse lightweight aggregate to identify the optimum substitution level of OPBC in OPS concrete.

3.2 Experimental Programme

3.2.1 Materials

3.2.1.1 Cement

Ordinary Portland cement (OPC) with a 7- and 28-day compressive strength of 36 and 48 MPa, respectively, was used. The specific gravity and Blaine specific surface area of the cement were 3.14 and 3510 cm^2/g, respectively.

3.2.1.2 Aggregate

The OPS and OPBC (Figure 3.1) used as coarse aggregate were collected from a local palm oil mill, then washed and dried in the laboratory. After drying, they were crushed using a crushing machine and then sieved. Table 3.1 shows that these two types of coarse aggregate have almost the same grading. For each mix proportion, the OPS and OPBC aggregates were weighed in dry conditions, immersed in water for 24 h, then air dried in the lab environment for 2-3 h to obtain an aggregate with an almost saturated dry surface condition. The physical properties of OPS and OPBC are shown in Table 3.2; it shows that OPBC is heavier than OPS but has significantly lower water absorption. The crushing, impact and abrasion values of OPBC are significantly higher than OPS, which shows that OPBC is weaker. In general, an OPBC grain is round in shape and has porosity on the surface, while, OPS is flaky without visible surface porosity (Figure 3.1).

Local mining sand was used as fine aggregate. It had a fineness modulus of 2.89, specific gravity of 2.68 and maximum grain size of 4.75 mm.

Figure 3.1: Oil palm shell (left) and oil-palm-boiler clinker (right)

Table 3.1: Grading of OPS and OPBC aggregates

Sieve size (mm)		19	12.5	9.5	8	4.75
Cumulative % by weight passing	OPS	100	96.80	84.24	61.20	2.98
	OPBC	100	98.35	90.32	70.75	3.27

Table 3.2: Physical and mechanical properties of aggregates

Physical and mechanical properties	Coarse aggregate		Fine aggregate
	OPS	OPBC	Normal Sand
Specific gravity (saturated surface dry)	1.19	1.69	2.68
Bulk density (compacted) (kg/m^3)	610	860	1657
24 h water absorption (%)	20.5	7.0	1.2
Crushing value (%)	0.2	21.2	-
Impact value (%)	5.5	36.3	-
Abrasion value (%)	5.7	23.9	-

3.2.1.3 Super-plasticizer (SP)

Sika ViscoCrete was used as the SP in this study. This admixture is chloride free according to BS 5075 and is compatible with all types of Portland cement including Sulphate Resistant Cement (SRC). It can be used at a rate of 500-2000 ml per 100 kg of cement, depending on workability and strength requirements. The maximum SP used in this study was 1% of total cement mass.

3.2.1.4 Water used

The water used was normal tap water. A fixed water to cement ratio of 0.36 was used in all mixes.

3.2.2 Mix Proportions

Six lightweight concrete mixes were prepared using OPS and OPBC as coarse aggregates; all mix proportions are shown in Table 3.3. The OPS concrete was considered the control concrete, and in all other mixtures, OPS was partially replaced with OPBC at 0, 10, 20, 30, 40 and 50% by volume. The cement content for all mixes was 480 kg/m^3, which is the same content used in most OPS concrete mixtures investigated in previous studies (Alengaram, Jumaat, & Mahmud, 2008a; Mannan & Ganapathy, 2001a, 2001b; Shafigh, Jumaat, & Mahmud, 2011).

Table 3.3: Mix proportions for concretes

Mix code	Content [kg/m^3]					SP (% cement)
	Cement	Water	Sand	Coarse aggregate		
				OPS	OPBC	
C-0	480	173	890	360	0	1
C-10	480	173	890	324	51	1
C-20	480	173	890	288	102	1
C-30	480	173	890	252	153	1
C-40	480	173	890	216	205	0.90
C-50	480	173	890	180	256	0.85

3.2.3 Curing Conditions

To determine the effect of the curing conditions on the 28-day compressive strength of the concrete mixes, specimens were cured under eight types of curing conditions, shown in Table 3.4. The curing symbols and their descriptions are listed below:

- FW: Specimens were immersed in water after demoulding until the age of testing.
- 3W: Specimens were cured in water for 2 days after demoulding and then air cured in the laboratory environment.

- 5W: Specimens were cured in water for 4 days after demoulding and then air cured in the laboratory environment.
- 7W: Specimens were cured in water for 6 days after demoulding and then air cured in the laboratory environment.
- 2T2D: Specimens were watered twice a day (morning and evening) for 2 days after demoulding and then air cured in the laboratory environment.
- 2T6D: Specimens were watered twice a day (morning and evening) for 6 days after demoulding and then air cured in the laboratory environment.
- PS: Specimens were wrapped in 4 layers of a 1 mm thick plastic sheet after demoulding and then kept in the laboratory environment.
- AC: Specimens were kept in the laboratory environment after demoulding.

Table 3.4: Curing conditions

Curing condition	Day in mould	Day in water	Watering (time/day)	Day in lab environment
FW	1	27	0	27
3W	1	2	0	25
5W	1	4	0	23
7W	1	6	0	21
2T2D	1	0	2	27
2T6D	1	0	2	27
PS	1	0	0	27
AC	1	0	0	27

3.2.4 Test Methods

To create each mix, the cement and aggregates were placed mixed for 2 min in a mixer. A mixture of 70% mixing water with SP was added, and mixing continued for another 3 min. The remaining water was added and mixing continued for another 5 min. After that, the slump test was performed. The concrete specimens were cast in 100 mm cube steel moulds to determine compressive strength, cylinders of 100 mm diameter and 200 mm height for splitting tensile strength, cylinders of 150 mm diameter and 300 mm height for modulus of elasticity, and prisms of 100 × 100 × 500 mm^3 for flexural strength. Specimens were compacted using a vibrating table.

After casting, specimens were covered with plastic sheets and stored in the laboratory environment, then demoulded one day after casting. Three test specimens were prepared to obtain average values of mechanical properties at any age. For curing condition, four specimens were used to obtain an average value. The main reason to prepare three or four specimens was to achieve the proper results of the property. The average values were only selected from those specimens which were giving 95-100% similar results.

To determine the water absorption of all mixes, specimens were dried in the oven at 105 ± 5 ^oC to reach a constant mass, then fully immersed in water kept at 23 ± 3 ^oC. Water absorption was measured after 30 min, then after 24 and 72 h.

3.3 Results and Discussion

3.3.1 Slump

The slump values of all the mixes are shown in Table 3.5. Since OPBC is round in shape and has a lower water absorption rate than OPS (about 66%), partial substitution of OPS by OPBC offers better workability. For example, mix C-30 showed a 39% higher slump value than OPS concrete for the same amount of SP. A structural LWAC with slump value in the range of 50-75 mm is considered to be a lightweight concrete with good workability (Mehta & Monteiro, 2006). Excessive slump value causes segregation of the LWA from the cement matrix. Therefore, to avoid segregation, SP content in the C-40 and C-50 mixes was reduced. However, these mixes had a much better slump value than the control mix, even though they had lower SP content of 10% and 15%, respectively. Increasing workability and using less SP are significant advantages of OPBC in OPS concrete.

Table 3.5: The slump value of all the mix proportions

Mix Code	Slump (mm)
C-0	55
C-10	58
C-20	65
C-30	90
C-40	100
C-50	90

3.3.2 Density

The unit weight of all the concrete mixes was measured 24 h after casting (immediately after demoulding), which is referred to as the demoulded density. The oven dry density was measured at 28 days. Table 3.2 shows that OPBC density was about 40-45% less than that of conventional coarse aggregate. However, compared to other types of lightweight aggregate such as OPS and coconut shell (both from agricultural waste), the density of OPBC is about 23% and 30% higher, respectively. The density of coarse OPBC is 6-46% higher than that of artificial LWAs such as LECA and Lytag, and 6-66% higher than that of natural LWAs like pumice, diatomite and volcanic cinders. Therefore, as expected, by substituting OPS with OPBC, concrete density increased. The relationship between density and OPBC content is shown in Figure 3.2. When OPS was replaced by OPBC, the demoulded density increased by around 2-4% compared to the control concrete.

Although the use of OPBC in OPS concrete increases its density, even at a 50% replacement level, density is still in the acceptable range for structural lightweight concrete. The C-0 and C-50 mixes were at about 20% and 24% replacement levels, respectively, which is lighter than conventional concrete. If oven dry density is considered, all concrete mixtures were lighter than the demoulded density by approximately 100-140 kg/m^3. This density for OPS crushed, OPS uncrushed and scoria lightweight aggregate concrete was 70-120 kg/m^3, 85-126 kg/m^3 and 82-124 kg/m^3, respectively (Kilic, Atis, Yasar, & Ozcan, 2003; Shafigh, Jumaat, & Mahmud, 2011; Shafigh, Jumaat, Mahmud, et al., 2011).

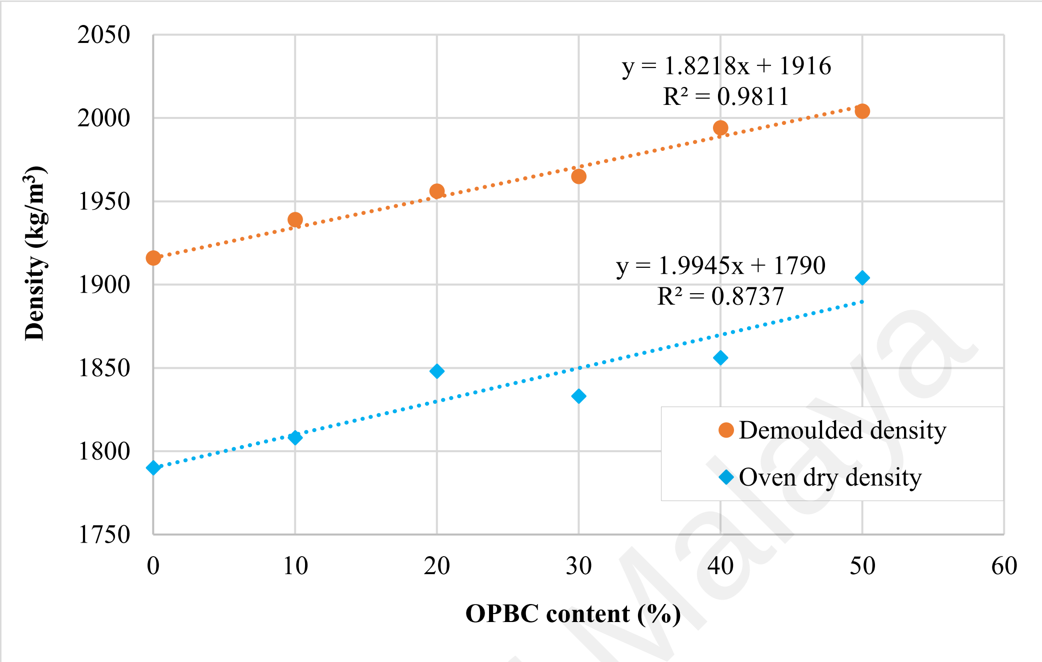


Figure 3.2: Relationship between density and OPBC content

3.3.3 Compressive Strength

3.3.3.1 Under continuous moist curing

The 28-day compressive strength of OPS concrete without OPBC aggregate (C-0) was about 36 MPa, which shows that the control mix is a normal strength lightweight concrete. The effects of OPBC aggregate addition on the compressive strength development of all mixes up to 56 days is shown in Figure 3.3. As can be seen in this figure, using 10% OPBC did not affect the compressive strength of OPS concrete. However, compared to the control mix, the compressive strength of the OPS specimens containing more than 10% OPBC improved significantly, particularly at later ages. Compressive strength of specimens containing 20-50% OPBC increased by 13-18%. The highest 28-day compressive strength, which was about 44 MPa was achieved for OPS concrete containing 30% OPBC.

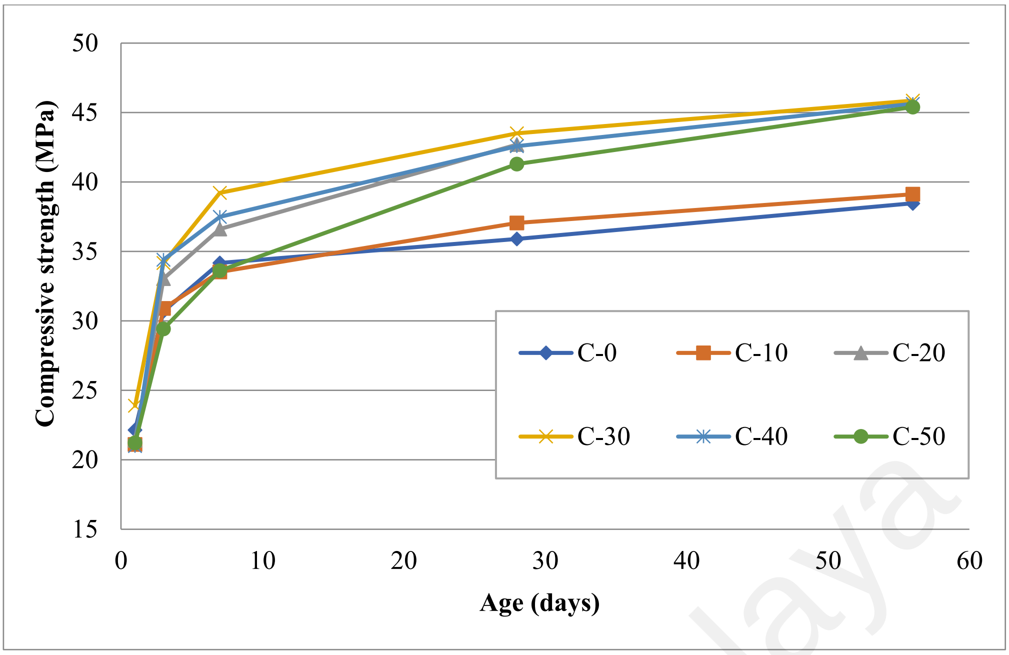


Figure 3.3: Compressive strength development of concrete

The compressive strength of most OPS concrete types was within the normal range of structural LWC. The strength of structural LWAC depends on the strength of the LWA used and on the hardened cement paste, as well as the bonding of the aggregate and cement paste in the interfacial zone (Lo, Tang, & Cui, 2007). Mannan, Basri, Zain, and Islam (2002) stated that the failure of OPS concrete in compression occurs due to failed adhesion between the OPS and the cement paste. They studied various types of pre-treatment approaches to improve the quality of OPS aggregate and the highest 28-day compressive strength they achieved was about 33 MPa. Okpala (1990) also studied the failure mechanism of OPS concrete and reported that failure was dependent on the breakdown of the bond between the aggregate and the cement paste. He observed that failure was conditional upon the combination of the failure of the shell and aggregate paste interface. Due to the smooth nature of the OPS aggregate on both the convex and concave faces, the bond between the shells and cement paste was not strong enough to sustain high loads (Aslam et al., 2015; Mannan et al., 2002; Okpala, 1990; Shafigh et al., 2011). Hence, the concrete failure was mainly due to debonding between the shells and cement paste, and the aggregates did not fully utilize their potential strength. On the other hand, OPBC is a porous aggregate with a rough surface texture (Figure 3.1). It was therefore expected that it would have a better interlock with the cement matrix, and, consequently, higher compressive strength. Figure 3.4 shows a section of concrete containing OPS and OPBC aggregate, and their connection with the cement matrix.

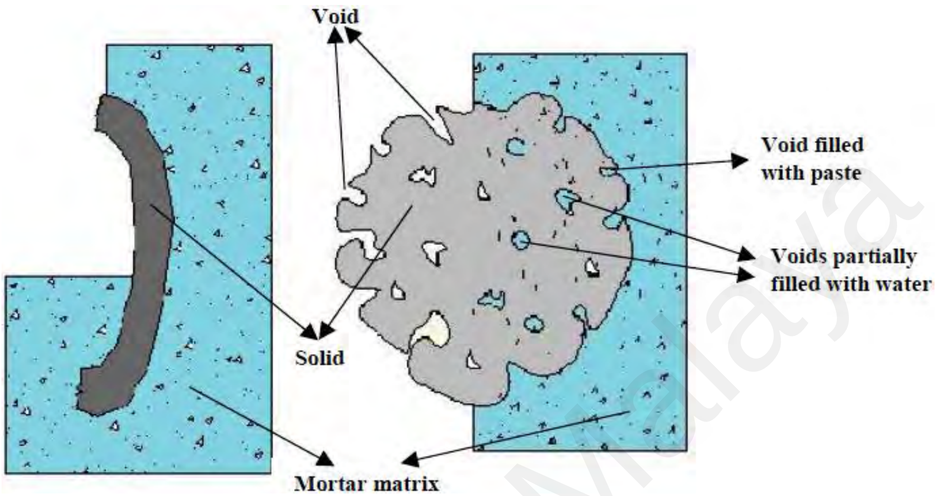


Figure 3.4: Surface texture of OPS and OPBC aggregates with mortar matrix

The 28-day compressive strength test results show that incorporating 20-50% OPBC in OPS concrete enhances compressive strength from grade 35 to grade 40, which can be considered high strength lightweight concrete. As reported earlier, the C-0 and C-10 mixes had almost the same compressive strength at all ages; 90-95% of the 28-day compressive strength was achieved at 7 days, while the same ratio for mixes containing 20-50% OPBC was 81-90%. For an artificial lightweight aggregate, this ratio was reported to be in the range of 76-87% (Wilson & Malhotra, 1988). The ratio for the 1-day and 3-day to 28-day compressive strength of OPS-OPBC concrete was in the range of 49-57% and 71-83%, respectively.

3.3.3.2 Under air-drying

All the concrete mixes showed a reduction on the compressive strength at 28 days under air-drying conditions (AC). Compared to the full water curing condition (FW), the reductions in compressive strength of the OPS concrete containing 0, 10%, 20%, 30%, 40% and 50% OPBC were about 25%, 10%, 17%, 9%, 3% and 3%, respectively. The reduction for the OPS-OPBC concrete was 3-17%, with an average value of about 8.5%. The contribution of OPBC significantly reduced strength loss in poor curing conditions, particularly when the contribution was more than 30%. In the AC condition, the OPS-OPBC concretes had 24-54% higher compressive strength than the control mix (C-0), while this range was 3-21% under FW curing.

One of the reasons for these results is the better performance of OPBC in terms of internal curing. During the process of internal curing, saturated lightweight aggregates develop suction pressure in the hydrated cement paste due to self-desiccation and chemical shrinkage. Henkensiefken, Bentz, Nantung, and Weiss (2009) reported that in the process of internal curing, the biggest pores will lose water first as the developed capillary stress decreases when the pores are emptied in this order. The LWA pores are generally larger than those of the surrounding cement paste, which increases cement hydration and strength development, and restrains shrinkage cracking behaviour. The pore sizes of OPBC aggregate are larger than those of OPS, and, hence, the OPBC aggregate quickly loses water and minimizes capillary stress when pores are emptied consequently, its internal curing performance is superior to that of OPS.

The replacement of normal coarse aggregates by LWA of similar size in high performance concrete as a means of providing internal curing for improved strength was studied by Weber & Reinhardt (1997a, 1997b); they named this method autogenous curing. In their study, months of continuous hydration after casting was observed with X-ray diffraction; the prepared concrete improved mechanical properties regardless of curing condition. They proposed a mechanism of water transport from the LWA to the hydrating cement paste based on capillary suction. In addition to the benefit of using LWA for increased compressive strength due to internal curing, it also enhanced durability performance due to the higher degree of hydration, improved density of the hydrated cement paste and reduced drying shrinkage, which minimized the effect of self-desiccation and elimination of plastic and drying shrinkage cracking (Shafigh et al., 2014; Weber & Reinhardt, 1997b).

Figure 3.5 shows the relationship between the 28-day compressive strength of all different types of OPS-OPBC concretes in air-drying and water curing conditions. As a comparison, this relationship was plotted for different types of concrete reported in previous research (Atis et al., 2005; Shafigh, Alengaram, Mahmud, & Jumaat, 2013; Shafigh, Jumaat, et al., 2013; Shafigh, Mahmud, et al., 2012). As seen in this figure, OPS concrete containing OPBC has a better compressive strength than both OP and normal weight concrete containing silica fume as well as OPS concrete with fly ash or GGBFS.

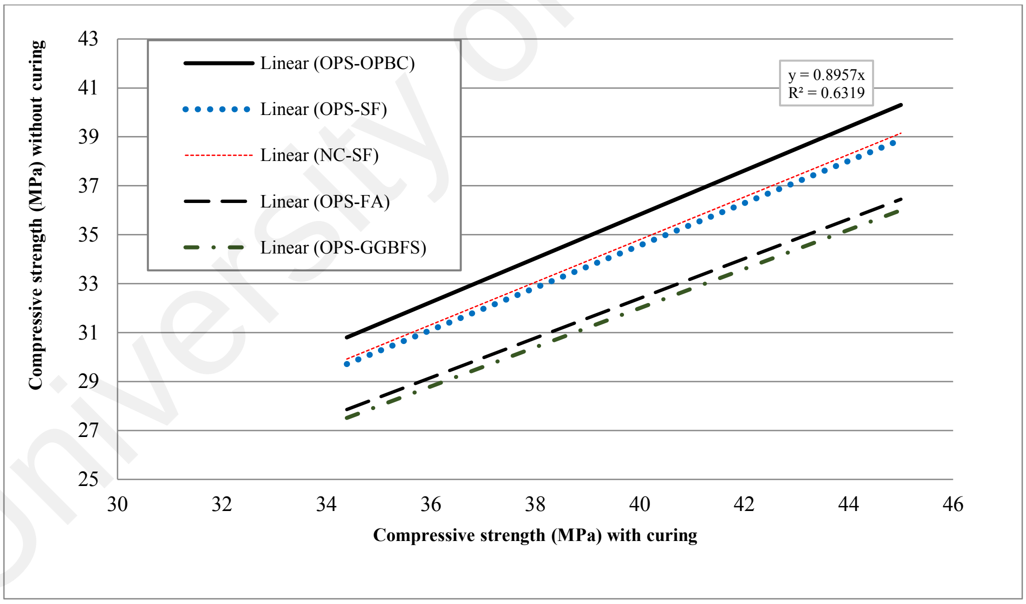


Figure 3.5: The relationship between compressive strength of OPS-OPBC concrete with and without curing and comparison with OPS concrete containing silica fume (OPS-SF), normal concrete containing silica fume (NC-SF), OPS concrete containing fly ash (OPS-FA) and OPS concrete containing GGBFS (OPS-GGBFS)

Source: (Atis et al., 2005; Shafigh, Alengaram, et al., 2013; Shafigh, Jumaat, et al., 2013; Shafigh, Mahmud, et al., 2012)

3.3.3.3 Under partial early curing

Curing is the practice of maintaining suitable moisture content and temperature in concrete during its early stages to ensure it develops its desired properties (ACI308, 1980). A minimum period of 7 days of moist curing is generally recommended with concrete containing normal Portland cement. However, for concrete mixtures containing a mineral admixture, a longer curing period is desirable to ensure the strength contribution from the pozzolanic reaction (Mehta & Monteiro, 2006). LWAs in concrete mixtures contain internal water, which helps increase the hydration of cement; due to the advantages of internal curing, recommended early age curing may be reduced in the case of lightweight aggregate concrete, which reduces the curing cost.

Table 3.6 shows the effect of different curing conditions on the 28-day compressive strength of all mixes. The data shows that partial early curing can improve 28-day compressive strength more than air-drying. Compared to the AC condition, the effectiveness of the 2T2D and 2T6D conditions on the improvement of compressive strength of OPS containing 0-20% OPBC was more significant compared to other mixes. Under these two curing conditions, the compressive strength of OPS concrete (mix C-0) was still significantly lower than for the FW curing condition, while the compressive strength of OPS concretes containing OPBC was very close to the FW curing condition. This is additional evidence that the compressive strength of concrete containing OPS as coarse aggregate is very sensitive to poor curing conditions. However, even a small amount of OPBC could significantly reduce this sensitivity. In general, partial early curing of 2T2D was better than 2T6D; if the curing method is a type of watering that occurs a few times a days, it should not continue over several days due to the negative effect of the wetting-drying condition on concrete. The test results of these two curing conditions reveal that 2T2D was better than 2T6D, with a savings in curing costs.

Compared to the AC condition, partial early curing of 2D, 4D and 6D improved compressive strength. A longer curing time resulted in better compressive strength for all concrete types. However, the rate of improvement for the OPS control mix was higher than for the other types, which may be due to the C-0 mix experiencing a significant reduction in compressive strength under the AC condition compared to other mixes. Early water curing therefore caused a greater improvement in compressive strength. However, the compressive strength of mix C-0 under 2D, 4D and 6D curing conditions was about 82-89% of compressive strength under FW curing, while this range for all types of OPS-OPBC concretes was 88-109%. This is additional evidence that concrete containing OPS as coarse aggregate needs special attention in terms of curing.

As can be seen in Table 3.6, the 28-day compressive strength for all concrete types under 2D, 4D and 6D conditions is almost the same. However, among the OPS-OPBC mixes, the closer values are for the C-40 and C-50 mixes which means that the 2D initial water curing may be sufficient for curing these two types of concrete.

Test results show that curing with a plastic sheet (PS) is a more effective method of improving compressive strength than air drying. This method led to an improvement of about 21% in the control mix (C-0), which was significantly higher than its effect on the OPS concrete containing OPBC, which was 3-12%. The higher improvement in the compressive strength of the control mix was due to a significant reduction (about 25%) in the compressive strength of this concrete under dry conditions. For OPS-OPBC concrete this drop was about 8%, on average. Any partial early curing therefore had better performance on the control mix compared to the OPS-OPBC concretes. If all types of partial curing conditions are considered, the control OPS concrete had about 13% more compressive strength than the AC condition and 6.5% for the OPS-OPBC concrete specimens.

Most codes of practice recommend 7-day moist curing (Haque, 1990). However, for some structural elements such as columns, it can be difficult to achieve. Nevertheless, as can be seen in Table 3.6, compressive strength is equivalent for 6D and PS, which means a plastic sheet can be used instead of 7-day moist curing; this method of curing may be more practical.

Table 3.6: Effect of different curing conditions on 28-day compressive strength

Mix codes	AC	2D	4D	6D	2T2D	2T6D	Plastic (PS)	FW
C-0	27.0	29.6	31.0	32.1	30.8	27.4	32.7	36.0
C-10	33.5	36.9	36.7	33.9	37.3	38.3	37.6	37.1
C-20	35.6	37.7	39.3	43.6	41.3	37.0	37.2	42.7
C-30	39.8	41.3	38.2	42.7	41.2	41.3	41.9	43.5
C-40	41.5	41.9	43.5	43.8	41.0	40.4	42.7	42.6
C-50	40.0	45.2	42.2	44.8	42.5	38.9	43.9	41.3

3.3.4 Splitting Tensile Strength

Splitting tensile strength for all the mixes is shown in Table 3.7. The minimum 28-day splitting tensile strength required for structural lightweight concrete to be used in structural elements is 2.0 MPa (Kockal & Ozturan, 2011). Table 3.7 shows that all the concrete mixes have more than 2.0 MPa splitting tensile strength from a 3-day age and that splitting tensile strength increased with compressive strength. The substitution of OPS with OPBC up to 30% did not affect splitting tensile strength at all ages (slightly reduced). The reduction was significant in the 40% and 50% substitution levels at early ages. However, it diminished with the increasing age of the concrete; the 28-day splitting tensile strength of the concretes containing 40% and 50% OPBC was similar to the control mix. It is interesting to note that a significant improvement was observed from 7 to 28 days for the splitting tensile strength of the C-40 and C-50 mixes, while improvement for the other mixes was minor. On the other hand, compressive strength improvement with water curing from 7 to 28 days was also significant, particularly for mix C-50.

The 28-day splitting tensile strength of concretes containing OPBC ranged from 3.05-3.31 MPa. Generally, the ratio of splitting tensile/compressive strength of normal weight concrete falls in the range of 8-14% (Kockal & Ozturan, 2011; Shafigh et al., 2014). However, the ratio of splitting tensile to compressive strength achieved for OPS concrete was 8-10% and 7-10.5% for OPS-OPBC mixes.

Table 3.7: Splitting tensile strength and flexural strength

Mix no.	Splitting tensile strength (MPa)				Flexural strength (MPa)	
	1 day	3 days	7 days	28 days	7 days	28 days
C-0	2.20	2.89	3.03	3.29	4.43	5.38
C-10	2.20	2.80	3.25	3.30	4.50	4.48
C-20	2.00	2.81	2.92	3.05	4.17	4.98
C-30	2.03	3.10	3.12	3.25	4.80	5.37
C-40	1.80	2.48	2.64	3.31	4.54	5.30
C-50	1.70	2.33	2.66	3.10	4.89	5.24

Figure 3.6 shows a comparison of splitting tensile strength results with those predicted by the equations proposed by various standards and researchers. ACI-318 (2005) proposed Eq. (3.1) for normal weight concrete with a cylinder compressive strength in the range of 21-83 MPa. Gesoglu, Ozturan, and Guneyisi (2004) proposed Eq. (3.2) for cold-bonded fly ash LWAC with a cube compressive strength ranging from 20 to 47 MPa and Eq. (3.3) is the proposed equation from (CEB/FIP, 1993). Shafigh et al. (2014) proposed Eq. (3.4) for LWC using two types of waste material from the palm oil industry. Neville (2008) reported Eq. (3.5) for pelletized blast furnace slag LWAC, with cube compressive strength ranging from 10 to 65 MPa.

$$f_t = 0.59\ (f_{cy})^{0.5} \qquad (3.1)$$

$$f_t = 0.27\ \sqrt[3]{f_{cu}^2} \qquad (3.2)$$

$$f_t = 0.301\ (f_{cy})^{0.67} \qquad (3.3)$$

$$f_t = 0.27\ (f_{cu})^{0.63} \qquad (3.4)$$

$$f_t = 0.23\ \sqrt[3]{f_{cu}^2} \tag{3.5}$$

where, f_t is the splitting tensile strength, f_{cu} and f_{cy} are the cube and cylindrical compressive strengths, respectively. As can be seen in Figure 3.6, Equations 3.2 and 3.3 showed values closer to the experimental results, with a reliability of about 90%.

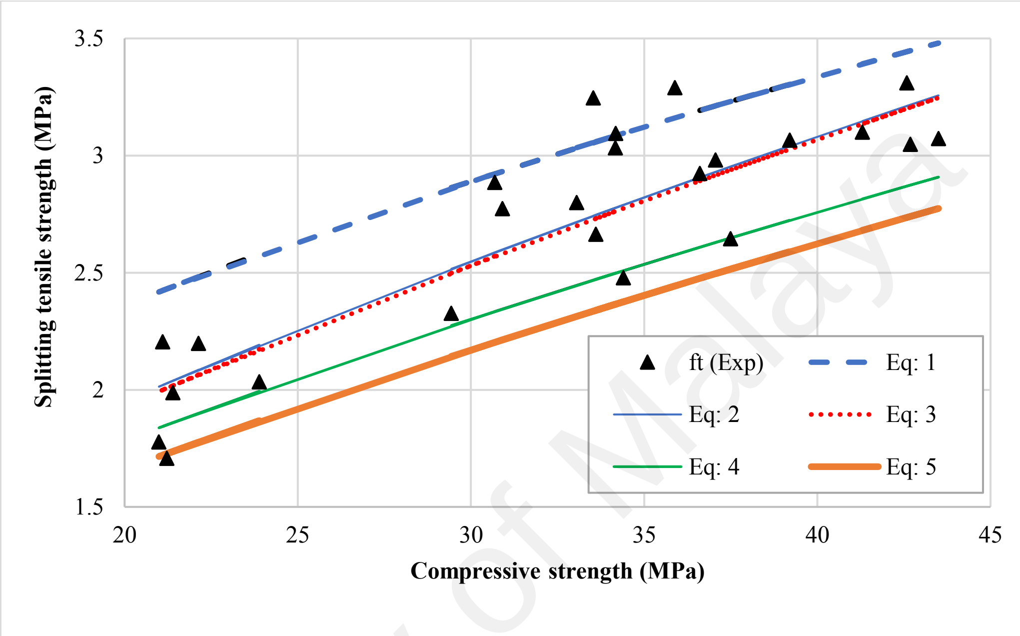


Figure 3.6: Experimental and theoretical splitting tensile strength of all concrete mixes

3.3.5 Flexural Strength

Flexural strength of all mixes at 7 and 28 days is shown in Table 3.7. The 28-day flexural strength of all OPS-OPBC mixes ranged from 4.48 to 5.52 MPa, an average of 12% higher than the 7-day flexural strength. The flexural strength of normal weight concrete with a compressive strength of 34-55 MPa is in the range of 5-6 MPa with a flexural/compressive strength ratio of 11.6-13.5% (Mehta & Monteiro, 2006). The 28-day flexural to compressive strength ratios of the OPS-OPBC mixes were 12-12.7%. It can therefore be concluded that all the OPS-OPBC concrete mixes had a similar flexural strength, and flexural to compressive strength ratio was similar to the NWC of the same grade. Holm and Bremner (2000) reported that the flexural strength of high strength lightweight aggregate concrete (HSLWAC) was generally 9-11% of compressive strength. The results of the study being described in this paper showed that OPS-OPBC concretes had a higher flexural to compressive strength ratio than HSLWAC. The ratio of splitting tensile to flexural strength for C-10, C-20, C-30, C-40 and C-50 was 67%, 61%, 57%, 62% and 59%, respectively. However, for OPS concrete with compressive strength from 34 to 53 MPa, this ratio varied between 51% and 72% (Alengaram, Jumaat, & Mahmud, 2008b; Shafigh, Jumaat, Mahmud, et al., 2012) depending mostly on the amount of OPS in the mixture (Shafigh et al., 2012).

Figure 3.7 shows the relationship between flexural strength and corresponding compressive strength. The experimental results were compared with the predicted results using equations proposed by applicable standards and researchers. CEB-FIP (1977) proposed Eq. (3.6) for LWC made with expanded shale and clay aggregates, with cubical compressive strength ranging varying between 20-60 MPa. Shafigh, Jumaat, Mahmud, et al. (2012) proposed Eq. (3.7) for crushed OPS concretes, with cubical compressive strength between 35 and 53 MPa. Zhang and Gjvorv (1991) proposed Eq. 3.8 to predict the flexural strength of high strength lightweight concrete. The equation prediction of Lo et al. (2004) is for expanded clay LWAC with a cubical compressive strength of 29-43 MPa Eq. (3.9).

$$f_r = 0.46\sqrt[3]{f_{cu}^2} \quad (3.6)$$

$$f_r = 0.12\, f_{cu}^{1.03} \quad (3.7)$$

$$f_r = 0.73\sqrt{f_{cu}} \quad (3.8)$$

$$f_r = 0.69\sqrt{f_{cu}} \quad (3.9)$$

where, f_r is the flexural strength, and f_{cu} is the cube compressive strength of the concrete in MPa. Equations 3.6 and 3.8 showed that predictions of flexural strength from cubical compressive strength were very close to the experimental results, with a reliability of about 99%. Equation 3.9 gave a more conservative estimate.

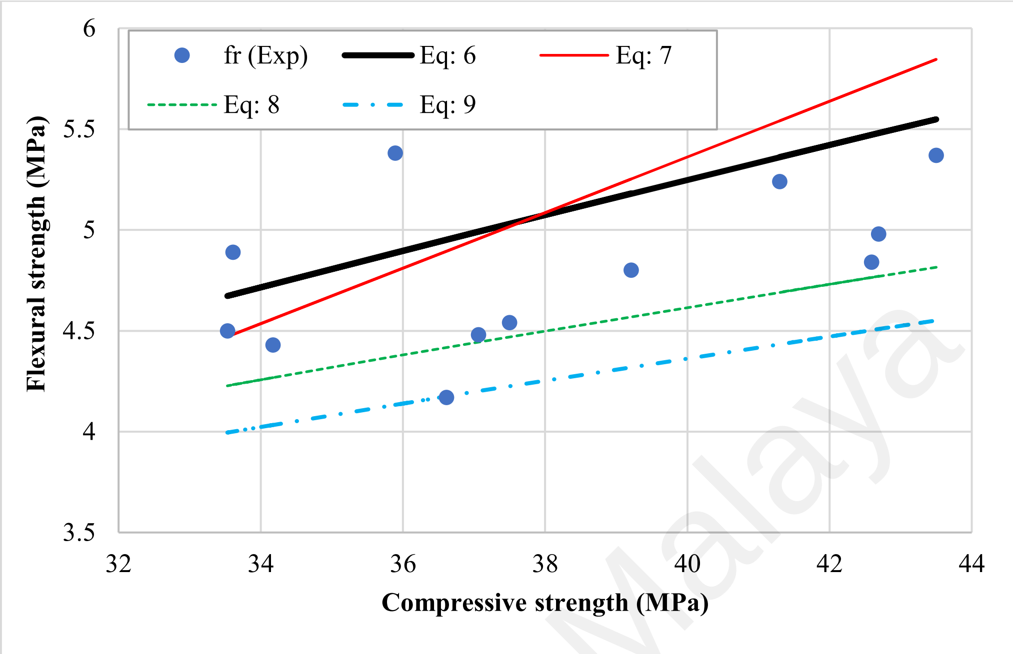


Figure 3.7: Experimental and theoretical flexural strength results of all concrete mixes

3.3.6 Modulus of Elasticity

The modulus of elasticity of the C-0, C-10, C-20, C-30, C-40 and C-50 mixtures was 7.9, 9.6, 10.2, 11.7, 13.0 and 15.0 GPa, respectively. The modulus of elasticity value of OPS concrete was considered to be low, while the contribution of OPBC aggregate into OPS concrete (at 10%, 20%, 30%, 40% and 50%) significantly increased the modulus of elasticity by about 18%, 23%, 32%, 39% and 47%, respectively. The modulus of elasticity of concrete depends on the moduli of elasticity of its components and their proportions by volume (Neville, 1971). The main difference between all the mixes was the type and volume of coarse aggregates; it can therefore be concluded from the modulus of elasticity test results that the modulus of OPBC grain is more than the modulus of OPS grain. On the other hand, the crushing, impact and abrasion values of OPBC are significantly less than the OPS as shown in Table 3.1.

Generally, the modulus of elasticity of structural LWC ranges between 10 and 24 GPa, and from 14 to 41 GPa in NWC (Shafigh et al., 2014). The test results of this study showed that the modulus of elasticity of OPS concrete containing OPBC aggregates was in the normal range for structural lightweight aggregate concretes.

Tasnimi (2004) presented Eq. (3.10) for artificial LWA concretes with a cylindrical compressive strength of about 15-55 MPa. Hossain, Ahmed, and Lachemi (2011) proposed Eq. (3.11), reporting data for LWC by incorporating pumice with a 28-day cylinder compressive strength of 16-35 MPa and density of about 1460-2185 kg/m^3. Alengaram et al. (2011) reported Eq. (3.12) for OPS concrete with 28-day cubical compressive strength of 25-39 MPa and air-dry density of 1640-1890 kg/m^3. The CEB/FIP model code (Short, 1978) proposed Eq. (3.13) to predict the modulus of elasticity of LWACs.

$$E = 2.1684\, f_y^{0.535} \quad (3.10)$$

$$E = 0.03 w^{1.5} f_y^{0.5} \quad (3.11)$$

$$E = 5\, f_{cu}^{0.33}\, (w/2400) \quad (3.12)$$

$$E = 9.1\, (w/2400)^2\, f_{cu}^2 \quad (3.13)$$

where, E is the modulus of elasticity (GPa), w is the dry density (kg/m^3), f_{cy} is the cylinder compressive strength (MPa) and f_{cu} is the cubical compressive strength (MPa).

When compared to test results, all the equations provided very conservative estimations with Eq. (3.12) being the closest to the experimental results. Changing the constant value in equation (Eq. 3.14) from 5 to 6 provided a good estimation with a reliability of about 90%.

$$E = 6\, f_{cu}^{0.33}\, (w/2400) \quad (3.14)$$

3.3.7 Water Absorption

Water absorption of all concrete mixes was measured at the age of 56 days for 30 min, 24 h and 72 h, as shown in Figure 3.8. The water absorption of the concretes containing OPBC aggregates was lower than that of the control OPS concrete. Water absorption was reduced when the amount of OPBC aggregate in the mixture was increased. Due to the high-water absorption of OPS, substitution of normal coarse aggregate with this lightweight aggregate increased its water absorption capacity. Therefore, reducing OPS volume in concrete was expected to reduce initial and final water absorption. Table 3.2 shows that the water absorption rate of OPBC aggregate was significantly lower than for OPS. Therefore, as shown in Figure 3.8, OPS-OPBC concretes had lower water absorption rates than OPS concrete. For OPS concretes with normal compressive strength, water absorption was higher than 10% (Teo et al., 2007). Water absorption for other types of structural lightweight concretes such as expanded polystyrene aggregate concrete and pumice aggregate concrete ranged from 3 to 6% and 14 to 22%, respectively (Babu & Babu, 2003; Gunduz & Ugur, 2005).

Ranjbar, Madandoust, Mousavi, and Yosefi (2013) categorized the quality of concrete as good, average and poor based on initial water absorption (absorption in 30 min) values of 0-3%, 3-5%, and above 5%, respectively. All concrete mixes showed initial absorption of less than 3%, which can be categorized as “good”. Moreover, the moderate (24 h) water absorption was in the range of 5.3-6.3% and the final (72 h) absorption was 6.3-7.1%. Neville (2008) reported that although concrete quality cannot be predicted by water absorption, good concretes generally have a rate of less than 10% by mass.

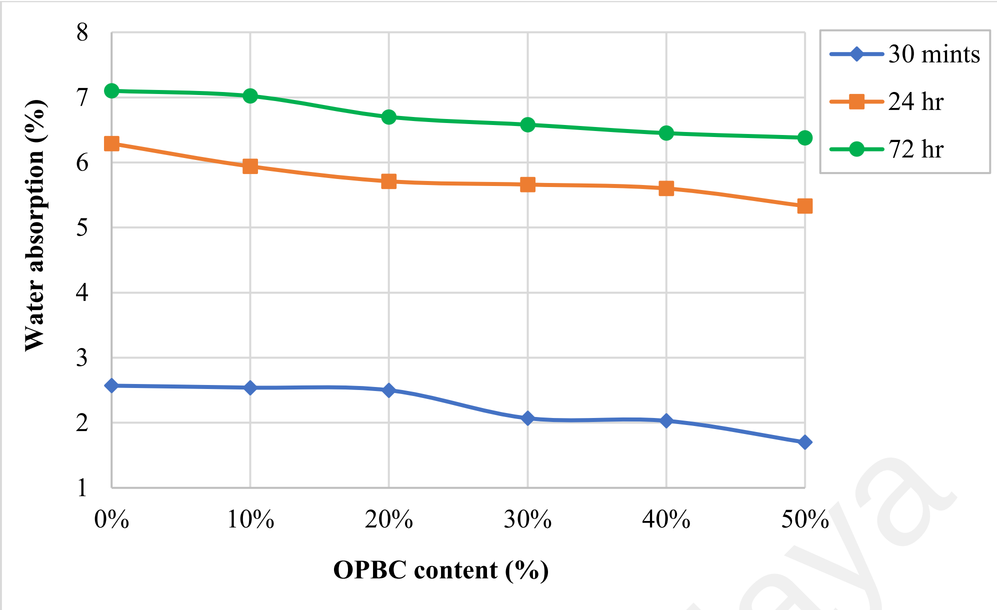


Figure 3.8: Relationship between OPBC content and water absorption

3.4 Conclusions

In this study, oil-palm-boiler clinker (OPBC) was partially used instead of oil palm shell (OPS) as coarse lightweight aggregate in an OPS lightweight concrete and the effect of this substitution on the mechanical properties of concrete was investigated. Based on the test results, the following conclusions can be drawn:

1. Increasing OPBC coarse aggregates in OPS concrete increased slump value.
2. Substituting OPS with OPBC aggregates increased the density of OPS concrete by about 2-4%. However, even at the 50% substitution level, density was still in the acceptable range for structural lightweight aggregate concrete.
3. The contribution of 20-50% levels of OPBC in OPS concrete significantly improved compressive strength under standard curing. At these substitution levels, grade 35 OPS concrete was transferred to grade 40, which can be considered high strength lightweight aggregate concrete. The optimum substitution level was determined as 30%.
4. Incorporating OPBC aggregate into OPS concrete reduces the sensitivity of compressive strength to lack of curing. OPS concrete (without OPBC aggregate)

under air-drying showed a reduction of about 25% compared to continuous water curing while OPS concrete containing 30-50% OPBC only showed a reduction of about 5%. This may be due to the better performance of OPBC lightweight aggregates for internal curing.

5. Partial early curing improves compressive strength more than air-drying because it is more effective when OPBC content is higher. However, if partial early curing occurs though watering a few times a day (like 2T2D and 2T6D), it is recommended that this method is not used continuously over several days due to the negative effect of the wetting-drying condition on concrete.
6. The 28-day splitting tensile strength of concretes containing OPBC was 3.05-3.31 MPa, which is in the usual range for structural lightweight aggregate concrete.
7. The 28-day flexural strength ranged from 4.48 to 5.38 MPa, which was 12-15% of the 28-day compressive strength. These ratios are equivalent to the normal weight concrete ratio.
8. The modulus of elasticity of OPS concretes increased about 18-24% with the incorporation of OPBC aggregates. OPS-OPBC concretes containing more than 20% OPBC aggregate had a modulus of elasticity in the normal range for structural lightweight aggregate concretes.
9. The water absorption of OPS concrete was reduced by increasing the percentage of OPBC aggregates. All OPS-OPBC mixes showed initial and final water absorption of less than 3% and 10%, respectively, and can be considered good concretes.

CHAPTER 4: HIGH STRENGTH LIGHTWEIGHT AGGREGATE CONCRETE USING BLENDED COARSE LIGHTWEIGHT AGGREGATE ORIGIN FROM PALM OIL INDUSTRY

4.1 Introduction

The concrete industry nowadays is the largest consumer of natural resources due to its widely usage in civil engineering structures. Its annually consumption of materials is as 2.282 billion tonnes of cement, 10-12 billion tonnes of stones and rocks together and 1 billion tonne of mixing water (Chuan, 2015; Mehta & Monteiro, 2006). Due to the huge amount of concrete production, it has a significant effect on the social, economic and environmental problems (Pelisser, Barcelos, Santos, Peterson, & Bernardin, 2012; Sari, Mat, Badri, & Zain, 2015; Tam, 2009). The best alternative to achieve an environmentally friendly and sustainability in concrete industry is to use waste and by-product materials instead of raw materials in concrete mixtures, which can contribute to a better quality of life for all mankind (Aslam et al., 2015; Shafigh, Jumaat, Mahmud, et al., 2012).

Lightweight concrete (LWC) is a most interesting field of research and has been widely used in buildings since ancient times. It has many advantages such as better heat insulation, sound absorption, fire and frost resistance and increased seismic damping (Aslam, Shafigh, & Jumaat, 2016b; Shafigh et al., 2010). High strength lightweight concrete can be produced up to grade 60 with an oven dry density range of about 350-2000 kg/m^3 (Shafigh et al., 2010). Study from Sari & Pasamehmetoglu (2005) revealed that the NWC has a low strength to weight ratio and its use in structural members such as multi story buildings, bridges and floating structures is a huge economic disadvantage. Therefore, the best way to resolve this issue is to use high strength lightweight concrete (HSLWC). It can also reduce the dead load of the structures by reducing the cross sections of beams, columns and foundations (Yasar, Atis, & Kiliç, 2004).

The most popular method of fabricating LWC is by using lightweight aggregate (LWA) (Polat et al., 2010). Since last few decades many types of lightweight aggregate such as foamed slag, volcanic cinders, diatomite, expanded clay, tuff, scoria, shale, slate, vermiculite, perlite and materials occurred as industrial by-products such as sintered slate, pulverized-fuel ash and expanded blast-furnace slag has been used as construction material (CEB/FIP, 1977; Neville, 2008; Shafigh et al., 2014). The artificial LWAs were produced by the application of high temperature and pressure, which results in high fuel costs (Shafigh et al., 2010). Therefore, the best alternative source for LWAs is the utilization of waste materials, which significantly reduces the construction costs as well as have many benefits in environmental issues. Oil palm shell (OPS) and oil-palm-boiler clinker (OPBC) are an alternative waste materials found in tropical regime countries and can be used as aggregate in concretes. It was found that Malaysia alone annually produced more than 4 million tonnes of OPS, very small amount of this solid waste is used as a fuel in oil palm mills (Sobuz et al., 2014). Previous studies (Hilmi, Shafigh, & Jumaat, 2014; Mannan & Neglo, 2010) revealed that OPS and OPBC can be used as coarse aggregate to produce green and sustainable structural lightweight aggregate concrete. The densities of the both aggregates were also found in the suitable range of structural lightweight aggregates. Recent studies (Aslam et al., 2015; Shafigh et al., 2014) have showed that the OPS and OPBC can be used as lightweight aggregate to produce high strength lightweight concrete.

Substitution of normal aggregates with OPS in normal weight concrete reduces its mechanical properties and increases water absorption of concrete (Shafigh et al., 2012). A lightweight concrete containing OPS has high drying shrinkage (Abdullah, 1996; Mannan & Ganapathy, 2002; Shafigh, Mahmud, et al., 2014). This concrete should have very well moist cured, otherwise it may have significant reduction on compressive strength (Mannan & Ganapathy, 2002; Shafigh et al., 2011). Higher volume OPS in concrete mixture causes to be more sensitive to lack of curing. On the other hand, OPBC is a crushed porous stone which is produced by burning of agricultural solid wastes in the boiler combustion process in oil palm mills. Therefore, it is expected that by using of OPBC in OPS concrete and reduction of the amount of OPS aggregates the engineering properties of resulted concrete are improved. Therefore, the objective of this paper is to produce a new type of lightweight aggregate concrete using a blended coarse lightweight aggregate by incorporating OPS and OPBC in concrete mixture. In this study, the engineering properties of the developed lightweight aggregate concrete were investigated. The mechanical properties such as compressive, splitting tensile and flexural strengths and modulus of elasticity of OPS-OPBC concrete were measured. In addition, initial and final water absorptions as well as drying shrinkage strain of this blended coarse lightweight aggregate concrete were also investigated. The drying shrinkage strain results were also compared by the ACI-Standard model.

4.2 Experimental Details

4.2.1 Materials Used

Ordinary Portland cement (OPC) with specific gravity of 3.14 and 28-day compressive strength of 48 MPa was used as binder. The OPS and OPBC were used as coarse aggregates (Figure 4.1). They were collected from a local palm oil mill then washed and dried in the laboratory. After drying, they were crushed using a crushing machine and sieved to achieve almost the same grading of the coarse aggregates (see Figure 4.2). For both concrete mixes, the OPS and OPBC aggregates were weighed in dry condition and then immersed in water for 24 hours. After that they were air dried in the lab environment for about 2 hours to obtain aggregate with saturated surface dry condition.

The physical and mechanical properties of OPS and OPBC are shown in Table 4.1. As can be seen from Table 4.1, OPBC has higher density but it has significantly lower water absorption compared to OPS aggregate. Local mining sand with a specific gravity of 2.68 and maximum grain size of 4.75 mm was used as fine aggregate. The super-plasticizer (SP) was selected according to ASTM C494-86 Type G with a density of 1.09 ± 0.02 kg/m^3.

Figure 4.1: Oil palm shell (left) and oil-palm-boiler clinker (right)

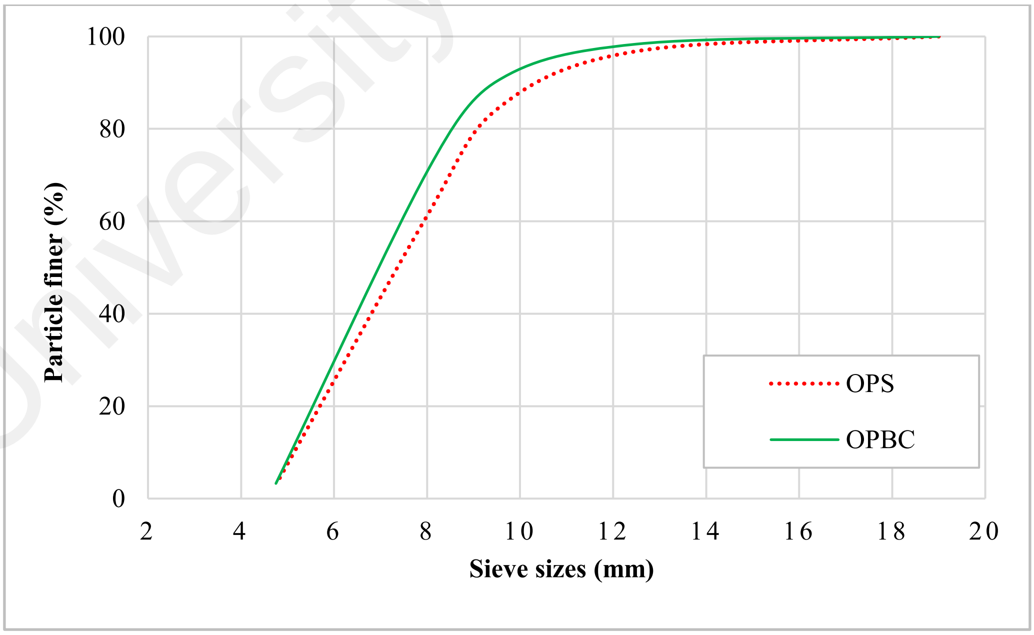


Figure 4.2: Grading of OPS and OPBC aggregates

Table 4.1: Physical and mechanical properties of lightweight aggregates

Physical and mechanical properties	OPBC	OPS
Specific gravity (saturated surface dry)	1.69	1.19
Compacted bulk density (kg/m^3)	860	610
24 h water absorption (%)	7.0	20.5
Abrasion value (%)	23.9	5.7
Crushing value (%)	21.2	0.2
Impact value (%)	36.3	5.5

4.3 Mix Proportions and Procedure

Concrete containing only OPS as coarse lightweight aggregate was considered as the control mix and in the other mix, 50% of volume of OPS was replaced with OPBC. The cement content and aggregates volume were placed constant for both mixes but the water content for OPS-OPBC mixture was reduced. The main reason for the reduction of water to cement ratio was to control the slump value of the concrete to maintain similar to slump of control OPS concrete. The mix proportions of both mixes are shown in Table 4.2.

Table 4.2: Mix proportions for concretes

Mix code	Content (kg/m^3)					w/c ratio	SP (% cement)
	OPC	Water	Sand	OPS	OPBC		
OPSC	480	173	890	360	0	0.36	1
OPBCC	480	139	890	180	256	0.29	1

For mixing, the cement and aggregates were placed into a mixer and mixed for 2 minutes. Subsequently, the mixture of SP and 70% of mixing water were added to the mixture and mixing was continued for another 3 minutes. After that the remaining water was added to the mixture and mixing was continued for another 5 minutes. Then the workability by performing the slump test was evaluated. Fresh concrete was then cast into 100 mm cube steel moulds for compressive strength, cylinders of 100 mm diameter and 200 mm height for splitting tensile strength, cylinders of 150 mm diameter and 300 mm height for elastic modulus, prisms of 100 × 100 × 500 mm^3 for flexural strength and prisms of 100 × 100 × 300 mm^3 for drying shrinkage strain tests. Specimens were compacted using a vibrating table in the laboratory with the temperature of 30 ± 2 ºC and relative humidity of about 70%. The specimens were demoulded after 24 hours of casting and were immersed in water at the temperature of 24 ± 2 ºC. Three test specimens were prepared to obtain the average values of the mechanical properties at any age. The drying shrinkage specimens were kept under 7 days moist curing, after that shrinkage readings were recorded under laboratory environment condition.

4.4 Results and Discussion

4.4.1 Workability and Density

Table 4.3 shows the slump value and density of the concrete mixes. It was observed that both OPSC and OPBCC mixes showed similar and satisfactory workability. As can be seen in Table 4.2, the water to cement ratio of OPBCC is significantly less than OPSC, while their workability was similar. This is due to the shape and physical properties of OPBC aggregate. Compared to OPS aggregates, OPBC aggregate is circular in shape and has lower water absorption of about 66%, therefore at the same mix proportions concrete containing OPBC has higher slump value compared to OPS concrete (Aslam, Shafigh, Jumaat, et al., 2016). For this reason, to keep constant similar slump value water content in OPBCC was reduced. According to Mehta & Monteiro (2006), structural lightweight aggregate concrete with a slump value in the range of 50 to 75 mm is considered as a LWC with good workability. At the same workability, concrete containing 50% coarse OPBC aggregates has about 20% lower water to cement ratio. This significant reduction in water to cement ratio may significantly affect hardened properties of the concrete.

Table 4.3 shows that concrete containing OPBC aggregate is heavier than OPS concrete. This is due to the density of OPBC is more than OPS. However, it should be noted that, although the incorporation of OPBC in OPS concrete increased the density, however, the density of OPBC concrete is still in the acceptable range for the structural lightweight concrete.

Table 4.3: Slump and density

Mix code	Slump (mm)	Density (kg/m^3)	
		Demoulded	Oven dry
OPSC	55	1920	1790
OPBCC	52	2015	1940

4.4.2 Compressive Strength

The development of compressive strength of OPSC and OPBCC mixes under continuous moist curing up to age of 56 days is shown in Figure 4.3. It was observed that both mixes showed similarly sharp gain in compressive strengths at early ages. The OPBCC mix has showed higher compressive strengths of about 25.9%, 28.1%, 28.4%, 30.7% and 28.9% at 1, 3, 7, 28 and 56 days, respectively compared to the control OPS concrete. The OPSC and OPBCC mixes showed a significant difference between the compressive strength results. The compressive strength of control OPS concrete showed a 11.4%, 16.8% and 20.2% increase at 7, 28 and 56 days, respectively, as compared to 3-days strength. While, these ratios for OPBCC mix are 11.7%, 19.8% and 21.2%, respectively. Consequently, the OPBC concrete mixture showed higher rate of strength gain up to 28-days but at later ages this rate was lower when compared to OPSC mix.

The OPBCC mixture showed 28-day compressive strength of about 53.3 MPa, which is about 30.7% higher than the control OPS concrete. The key problem in OPS concrete is due to it includes many shapes such as roughly parabolic, flaky and irregular with smooth surface texture. OPBCC showed higher compressive strength due to: 1) OPBC aggregates are not flakey. 2) They have rough surfaces which improve interlocking between cement matrix and aggregate. 3) The density of OPBC aggregate is higher than OPS aggregate, and, therefore, reduction of OPS and increase the OPBC content improved the compressive strength of concrete. 4) it was found that due to round shape of OPBC grains, substitution of OPS with OPBC improved the workability of concrete.

Therefore, for the same workability, the water to cement ratio in OPBCC could be reduced. The lower water to cement ratio increased the compressive strength.

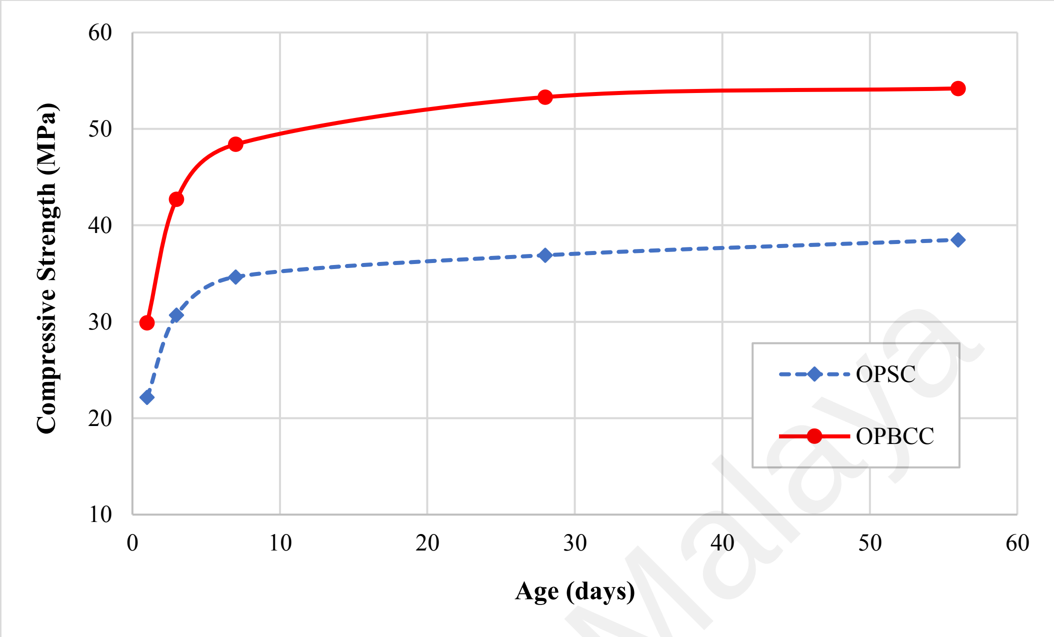


Figure 4.3: Development of compressive strength

Lo et al. (2004) reported that strength of LWC depends on the properties of the aggregates used and the hardened cement paste and their interfacial bonding. Okpala (1990) reported that the failure of OPS concrete dependent upon the breakdown of the bond between the shell and the cement paste. Mannan et al. (2006) stated that the OPS concrete was generally failed due to the adhesion between the shell and the cement paste. Further, they improved the quality of the shells using pre-treatment methods and achieved the 28-day compressive strength of about 33 MPa.

From 28 days compressive strength results it can be seen that the replacement of OPS by 50% OPBC in second mix highly increases the compressive strength from grade 35 to grade 50 which can be consider as high strength lightweight aggregate concrete. The major reason to show higher compressive strength in OPBCC is due to this concrete has significantly lower water to cement ratio as compared to OPS concrete. Therefore, it is expected that this aggregate has better interlock with cement matrix and consequently higher compressive strength.

4.4.3 Splitting Tensile and Flexural Strengths

ASTMC330 (2005) specified that the minimum 28-day splitting tensile strength required for structural LWAC must be 2.0 MPa. As can be seen in Table 4.4, both mixes showed significantly higher splitting tensile strength (39% for OPSC and 44% for OPBCC) than the minimum requirement of ASTMC330 (2005). Generally, the splitting tensile strength of the concretes is proportional to its compressive strength, the higher the compressive strength higher will be the splitting tensile strength. Literature (Abdullah, 1996; Alengaram et al., 2008a; Mannan & Ganapathy, 2002) revealed that under standard curing, the OPS concrete showed the 28-day splitting tensile strength in the range of 1.10-2.41 MPa. As can be seen in Table 4.4, in this study, OPS concrete showed significantly higher splitting tensile strength compared to previous studies. However, in the mix OPBCC, the replacement of OPS aggregate by 50% OPBC aggregate improved the splitting tensile strength at all ages. It is interesting to note that a significant improvement from 7 to 28 days for the splitting tensile strength of the OPBCC mix was observed, while for OPS concrete the improvement was small. Normally, the ratio of splitting tensile to compressive strength of NWC was found in the range of 8-14% (Shafigh et al., 2014). However, this ratio for OPS and OPBC concretes is about 9% and 6.7%, respectively. This ratio for OPS concrete was in the range of NWCs, however, this ratio for OPBCC was lower than the minimum value of 8%. This is due to OPBCC is a high strength concrete. In high strength LWC, this ratio is in the range of 6-7% (Holm & Bremner, 2000).

The flexural strength of OPSC and OPBCC mixes at 7- and 28 days are shown in Table 4.4. It was observed that similar to compressive and splitting tensile strengths, the flexural strength of OPBCC mix is also higher than OPS control mixture. In order, the OPBCC mix showed the compressive, splitting tensile and flexural strengths of about 31%, 23% and 8% higher than the control OPS concrete, respectively. Mehta and Monteiro (2006) reported that the flexural strength of NWC with a compressive strength of 34-55 MPa is in the range of 5-6 MPa, with flexural to compressive strength ratio of about 11.6-13.5%. However, the flexural to compressive strength ratio of high strength LWAC was generally varies in the range of 9-11% (Holm & Bremner, 2000). The 28-day flexural to compressive strength ratio of OPSC and OPBCC mixes was about 14.6% and 13.2%, respectively. The flexural to compressive strength ratio of OPBCC mix is similar to NWC and greater than high strength LWAC.

Table 4.4: Slitting tensile and flexural strengths

Mix code	Splitting tensile strength (MPa)				Flexural strength (MPa)	
	1 day	3 days	7 days	28 days	7 days	28 days
OPSC	2.20	2.89	3.03	3.29	4.43	5.38
OPBCC	2.59	3.00	3.25	3.58	5.15	7.00

4.4.4 Modulus of Elasticity

The modulus of elasticity plays very important role in civil engineering structures. It measures the material resistance to the axial deformation. It represents the maximum allowable stress limit of that material before undergoing the permanent deformation. Its value is obtained by measuring the slope of the axial stress-strain curve in the elastic region (Malesev, Radonjanin, Lukic, & Bulatovic, 2014). It was reported that the modulus of elasticity for structural lightweight concrete is 17-28 GPa while it is 20-40 GPa for normal weight concrete (Holm & Bremner, 2000). The modulus of elasticity of the OPSC and OPBCC mixes were 7.9 GPa and 15.7 GPa, respectively. The OPS concrete was considered as a concrete with low elastic modulus, while the replacement of OPS by 50% OPBC aggregates significantly increased the modulus of elasticity by about 50%. Neville (1971) reported that the modulus of elasticity of concrete depends on the moduli of elasticity of its components and their proportions by volume in the concrete. Kosmatka et al. (2002) stated that the modulus of elasticity of NWC ranges between 14 and 41 GPa, while, in LWC, it ranges from 10 to 24 GPa. The test results of this study showed that the modulus of elasticity of OPS concrete containing OPBC aggregates was in the range for NWC as well as structural lightweight aggregate concretes.

4.4.5 Water Absorption

Results of initial (30 mints) and final (72 hours) water absorptions are shown in Figure 4.4. CEB-FIP (Ranjbar et al., 2013) categorized concrete quality as poor, average and good for initial water absorption values of 5% and above, 3-5% and 0-3%, respectively. Both concrete mixes showed an initial water absorption of less than 3% which can be categorized as "good" quality concrete. Incorporating of OPBC in OPS concrete could reduce initial water absorption of OPS concrete about 45%. Final water absorption of OPBCC was about 42% less than OPSC. The reduction of water absorption is due to the lower water absorption of the OPBC aggregates compared to OPS aggregates (Table 4.1). Generally, the OPS concretes with normal compressive strengths showed water absorption higher than 10% (Teo et al., 2007). Neville (2008) reported that although the concrete quality cannot be predicted by the absorption of water, however in general, good concretes has a water absorption of less than 10% by mass. On the other hand, there is this believe that in most cases good concretes have final water absorption less than 5% (Kosmatka et al., 2002). By considering all the criteria's in water absorption of concrete, test results of this study show that OPBCC can be considered as good quality concrete.

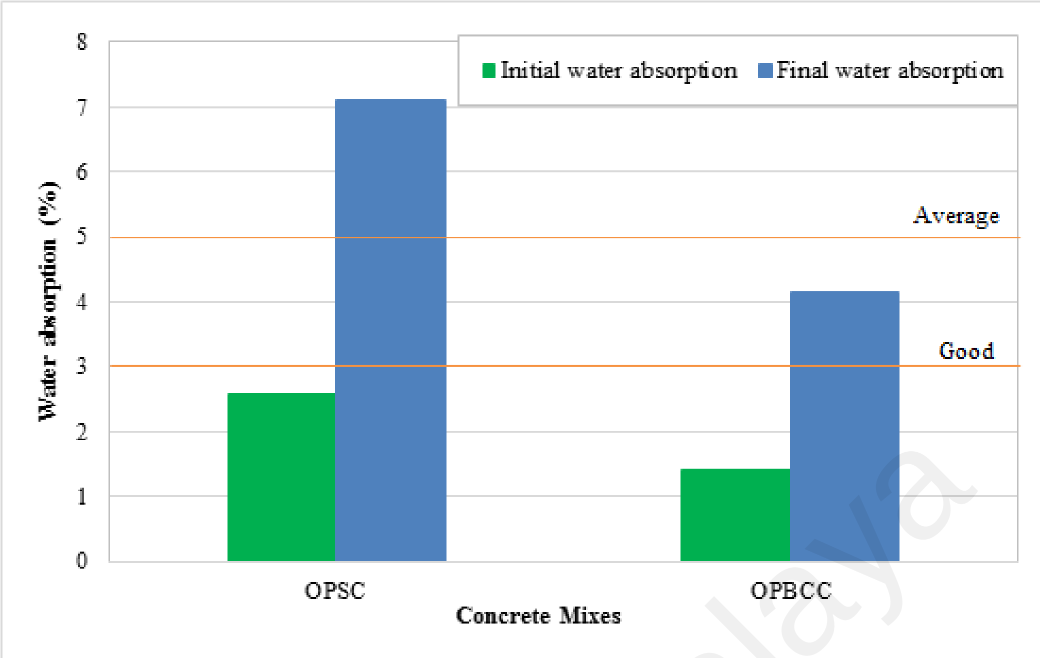


Figure 4.4: Water absorption of concrete mixes

4.4.6 Drying Shrinkage

The development of the drying shrinkage strain of the lightweight concretes (OPSC and OPBCC) after 7 days moist curing for up to about 8 months are showed in Figure 4.5. As can be seen in the figure, there is a significant difference between the shrinkage results of the both mixes. The control mixture (OPSC) showed the highest drying shrinkage strain of about 614 mm/mm which is about 36% higher than the OPBCC mixture. This shows that the substitution of OPS with OPBC aggregates has a significant influence on the drying shrinkage strain. Aslam, Shafigh, and Jumaat (2016a) investigated the drying shrinkage behaviour of structural lightweight aggregate concrete using blended oil palm bio-products. They reported that 7-day moist cured concretes containing both types of OPS and OPBC aggregates have lower drying shrinkage compared to structural lightweight aggregate concretes were made of lightweight aggregates such as lytag, expanded shale or sintered fly ash. Al-Khaiyat and Haque (1998) investigated the long-term drying shrinkage performance of lytag lightweight aggregate concrete after 7 days moist curing. In their study, the drying shrinkage of about 640 micro strain was achieved at the age of 3 months. They have reported that various types of lightweight aggregate usually resulted different behaviour in drying shrinkage. Further, some lightweight concretes made of expanded clay and expanded shale aggregates with the compressive strengths of 30-50 MPa showed a drying shrinkage strain in the range of 400 to 600 micro-strain (CEB/FIP, 1977).

The drying shrinkage strain of the OPS concrete was significantly reduced, might be due to the following reasons. (1) OPS is an agricultural waste with smooth surface texture and has lower specific surface area, while the OPBC is a porous crushed stone so it is expected that the drying shrinkage of OPS is higher than OPBC aggregate. Al-Attar (2008) investigated the shrinkage strain of NWC by using crushed and uncrushed gravels as aggregates. He reported that the round uncrushed gravels have lower specific surface area and smoother texture due to that it showed higher drying shrinkage compared to crushed gravel aggregate concrete. Therefore, it can be seen that surface texture of aggregate influences the drying shrinkage of concrete. (2) It was observed that by the incorporation of the OPBC in OPS concrete, the all mechanical properties were improved. This is another reason that concrete with higher elastic modulus have lower drying shrinkage. This might be due to the strong interfacial bond between the OPBC aggregates and the cement paste. Neville (1977) reported that LWA usually leads to higher shrinkage in concrete due to its lower elastic modulus. In this study, the modulus of elasticity was highly improved by the contribution of the OPBC aggregate and was found in the range of normal weight concretes. (3) Another important reason of lower drying shrinkage strain of OPBC concrete was due to significant reduction of water to cement ratio from 0.36 to 0.29 in mix OPBCC. Bogas, Nogueira, and Almeida (2014) reported that for the same cement content, the drying shrinkage strain increases with increasing water to cement ratio. In fact, there is an increment of the volume of paste and a corresponding reduction of the aggregate content. The higher the water to cement ratio the lower the mortar stiffness and the higher the volume of evaporable water. A small reduction of the w/c ratio also causes a significant delay in drying shrinkage strain (Carlson, 1938).

The experimental results of the drying shrinkage strain were also compared with the prediction model proposed by ACI209R (2008). The ACI209R (2008) proposed shrinkage prediction model as, $S_{(t,\ tc)}$ at time t (days) measured from start of drying at t_c (days), and S_∞ is the ultimate shrinkage.

$$S_{(t,\ tc)} = \frac{(t - t_c)}{f + (t - t_c)} \text{ x } S_\infty \tag{4.1}$$

$$f = 26.0 * e^{\left[1.42*10^{-2}\left(\frac{V}{S}\right)\right]} \tag{4.2}$$

$$S_\infty = 780 \text{ x } 10^{-6} \text{ x } (Y_{sh}) \tag{4.3}$$

$$Y_{sh} = Y_{tc} \cdot Y_{RH} \cdot Y_{vs} \cdot Y_s \cdot Y_\psi \cdot Y_c \cdot Y_\alpha \tag{4.4}$$

where, f = 35 for concrete water-cured for 7 days while to take into account the size and geometry on the drying of concrete specimens. Volume to surface area ratio is $\left(\frac{V}{S}\right)$ and Y_{sh} represents the product of several factors. Y_{tc} is the curing time coefficient, Y_{RH} is the relative humidity coefficient, Y_{vs} depends on volume to surface area ratio, Y_s is the slump factor (slump in mm), Y_ψ is the fine aggregate ratio (fine aggregate to the total aggregates), Y_c is the cement content in kg/m^3 and Y_α is the air content (%).

The ACI209R (2008) prediction model of shrinkage strain gave close results to shrinkage values of both concretes up to one month. After one month, the drying shrinkage values from the model are closer to drying shrinkage of OPSC mix. In general, it can be concluded that this model code is suitable to predict drying shrinkage of OPS and OPS-OPBC concretes at short time (for up to one month) and for OPS concrete at later ages (for more than five months). Therefore, a new model code to predict drying shrinkage of OPS-OPBC concrete should be developed.

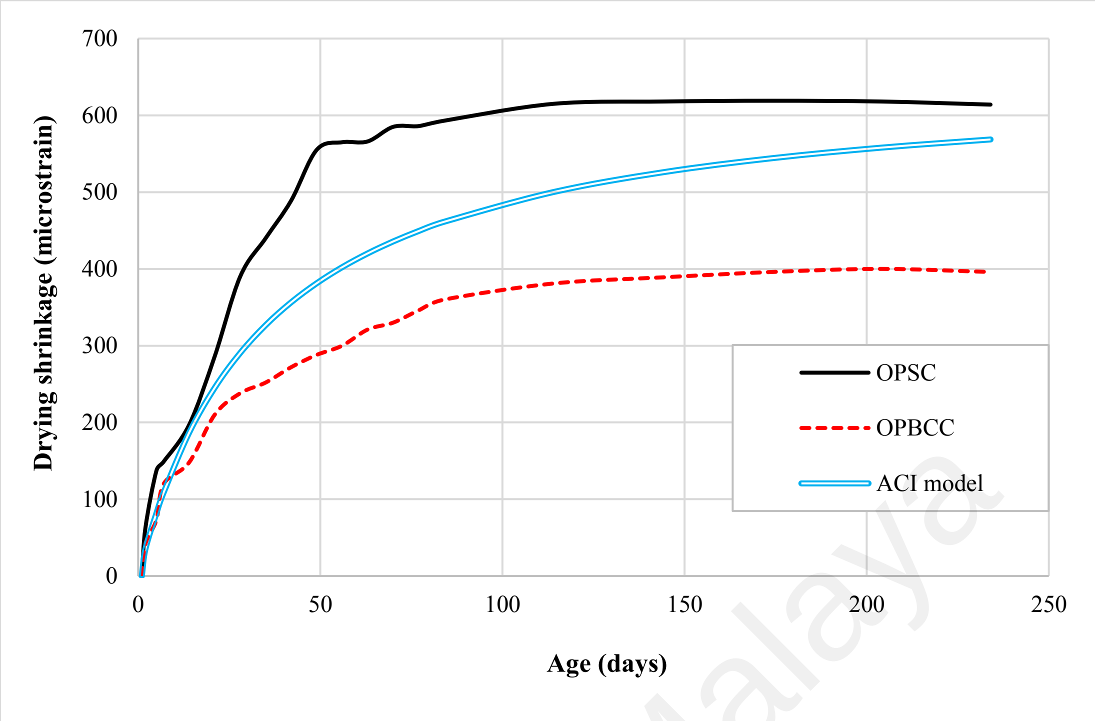


Figure 4.5: Drying shrinkage strain development of concretes

4.5 Conclusions

In this study, the mechanical and engineering properties of high strength lightweight aggregate concrete using blended coarse lightweight aggregates were investigated. From test results, the following conclusions can be drawn:

1. Due to the round shape of OPBC aggregates, incorporation of this aggregate in OPS concrete improves workability of the concrete.
2. Due to an OPBC grain is about 42% heavier than an OPS grain, inclusion of OPBC in OPS concrete increased the density of concrete. However, the density of OPS-OPBC concrete was still in the acceptable range for structural lightweight aggregate concretes.
3. The substitution of 50% OPS with OPBC in OPS concrete the compressive, splitting tensile and flexural strengths significantly improved. By this substitution, grade 35 concrete with the oven dry density of about 1800 kg/m^3 was transferred to grade 50 concrete with the oven dry density of about 1950 kg/m^3.

4. The modulus of elasticity of grade 35 OPS concrete is very low compared to normal concrete and structural lightweight aggregate concrete at the same compressive strength. However, the incorporation of OPBC in OPS concrete significantly enhanced this property. The modulus of elasticity of OPS-OPBC concrete is in the normal range of structural concretes.
5. The initial and final water absorption of OPS-OPBC concrete is significantly less than OPS concrete. Based on water absorption, this concrete is considered as good quality concrete.
6. The drying shrinkage of OPS and OPS-OPBC concretes is similar at early ages. However, OPS-OPBC concrete showed significantly lower drying shrinkage compared to OPS concrete after one month. The ACI-209R has conservative estimation for OPS-OPBC concrete.

CHAPTER 5: MANUFACTURING HIGH-STRENGTH LIGHTWEIGHT AGGREGATE CONCRETE USING OIL PALM SHELL IN A SEMI-LIGHTWEIGHT OIL-PALM-BOILER CLINKER CONCRETE

5.1 Introduction

The weight of concrete is one of the most important parameters to make an economical structure. Compared to conventional concrete, lightweight concrete (LWC) shows better economy with lower dead load and higher efficiency (Shafigh, Jumaat, & Mahmud, 2012). Lightweight concrete has been used since ancient times and is a most interesting field of research because of its several advantages including lesser transport, reinforcement, and foundation cost, cost-effective scaffolding and formwork, improved constructability, better durability, no surface bleed water, sound absorption, superior anti-condensation properties, improved hydration due to internal curing, lower tendency to buckle due to variant temperature gradients, reduced seismic forces, and better heat insulation, fire and frost resistance (CEB/FIP, 1977; Duzgun, Gul, & Aydin, 2005; Neville, 2008). However, this type of concrete also has several disadvantages including lower mechanical and durability properties, greater amount of cement is required compared to conventional concrete of the same grade, higher drying shrinkage and creep, high prestressing losses, higher material costs, and lack of sufficient shear reinforcement (Shafigh et al., 2011). Such drawbacks justified the efforts to resolve the problems of the existing lightweight concretes.

The most popular way of achieving structural LWC is by using lightweight aggregates (LWA) (Polat et al., 2010), which may be either natural or artificial. The main natural LWAs such as diatomite, pumice, volcanic cinders, scoria, tuff and artificial LWAs such as expanded clay, shale, slate, perlite and vermiculite have been used as construction materials (Aslam et al., 2016b). Another type of artificial (factory-made) LWA is an oil-palm-boiler clinker (OPBC) obtained from the oil palm industry. In countries, such as Malaysia, Indonesia and Nigeria, the oil palm industry produces huge amount of agricultural wastes. Malaysia contributes about 58% of the total world supply of palm oil, and one of the top listed countries with huge amount of solid waste from the oil palm industry (Ahmad et al., 2010). The OPBC is produced during the oil palm extraction process and is locally known as boiler stone. It is also referred to as oil-palm-boiler clinker (OPBC) with whitish grey in colour and has the appearance of a porous stone, which contributes to its light-weightiness (Aslam, Shafigh, Jumaat, et al., 2016; Soleymani, 2012). The density of OPBC aggregates is about 15-35 % less than conventional aggregates.

Several researchers have studied the use of OPBC as an aggregate in concretes to achieve varying grades of strength. Some of them used OPBC as both fine and coarse aggregates to develop a LWAC known as oil-palm-boiler clinker concrete. Abdullahi et al. (2008) investigated the properties of LWC using OPBC as fine and coarse aggregates in concrete mixture. They successfully developed a LWC with 28-day compressive strength range of 17-33 MPa with the dry density in the range of 1440-1850 kg/m^3. Zakaria (1986) prepared the LWC by utilizing the OPBC as coarse and normal sand as fine aggregates. He achieved the 28-day compressive strength range of 15-28 MPa, with the dry density range of 1800-2000 kg/m^3. Later, Mannan and Neglo (2010) studied the use of OPBC as coarse aggregate in concrete to develop a structural LWAC. They achieved better compressive strength varying between 27-36 MPa with the dry density range of 1845-1980 kg/m^3. Ahmad et al. (2007) changed the concrete mixture by using OPBC as coarse and mining sand as fine aggregate. They produced semi-lightweight aggregate concrete with the 28-day compressive strength of about 42 MPa and the dry density of about 2020 kg/m^3.

From the previous studies, it was observed that using OPBC as coarse aggregate with normal sand, the density of concrete might be higher than the density required for a structural lightweight concrete according to code of practices (ACI213R, 1999). The density of the structural LWAC normally ranges from 1400-2000 kg/m^3 compared to that of normal weight concrete (NWC) that is around 2400 kg/m^3. Neville (2008) reported that the density of LWC lies between 350 to 1850 kg/m^3. In the most cases, artificial LWAs were used to produce structural LWAC in practice. However, it should be noted that manufacturing process of these types of LWAs is not ecological and environmental due to need of very high temperature up to about 1200 °C (Priyadharshini & Santhi, 2012). To provide such a high temperature huge amount of fossil fuel is required. In addition, the cost of artificial LWAs is more than conventional aggregates because of high energy demand for preparation process. If this type of LWA is not locally available, it should be imported which makes it to be more expansive. A solid waste namely oil palm shell (OPS) is an alternative LWA in tropical regions, where there is oil palm industry. OPS is an agro-waste material and is the most abundantly produced solid waste material in tropical regions. It has characteristics of a lightweight aggregate (Okafor, 1988) and can be used in concrete mixture to produce a structural LWAC (Mannan & Ganapathy, 2002; Shafigh, Jumaat, & Mahmud, 2012). OPS lightweight aggregate is about 58% lighter than conventional aggregates. OPS lightweight concrete with a 28-day compressive strength of 20-35 MPa (typical compressive strength range for structural LWC) has a dry density of about 20-25% less than conventional concrete (Mannan & Ganapathy, 2004). Recently, high strength OPS lightweight concretes with a compressive strength in the range of 42-53 MPa and dry density in the range of 1790-1990 kg/m^3 were also produced by (Shafigh, Jumaat, & Mahmud, 2012; Shafigh, Jumaat, Mahmud, et al., 2011).

From the literature, it could be found that OPBC concrete with high strength is considered as semi-lightweight concrete. In addition, reports show that the OPBC concrete is heavier than OPS concrete at the same compressive strength. This is because the density of OPBC is about 30% more than OPS. To reduce the density of OPBC concrete from semi-lightweight to lightweight concrete a part of OPBC volume should be substituted by a lighter lightweight aggregate. OPS with a density about 30% less than OPBC could be suitable to be used for this purpose. In the current study, OPBC therefore was partially substituted with OPS aggregates in a grade 55 semi-lightweight OPBC concrete.

This study is organized as follows: In section 5.2, we describe our experiment setup in details. In section 5.3, we evaluate the efficiency of OPS aggregates on density, compressive strengths under different curing conditions, splitting tensile, flexural strengths, modulus of elasticity, stress-strain relationship, and water absorption in a semi-lightweight OPBC concrete. Finally, we summarize our conclusions in section 5.4.

5.2 Experimental Investigation

5.2.1 Materials Used

The ordinary Portland cement (OPC) with the 28-day compressive strength of 48 MPa was used as a binder. The OPC conforming to MS522, part-1:2003 with a specific surface area and specific gravity of 1.89 m^2/g and 3.14, respectively.

Local mining sand with a maximum size of 4.75 mm and specific gravity of 2.68, was used as fine aggregate. The locally collected materials of OPBC and OPS, as shown Figure 4.1, were used as coarse aggregates. The OPS aggregates were washed with detergent to remove the dust and the oil coating from the surface and dried in the laboratory. However, the OPBC aggregates were crushed using a crushing machine and then sieved. The both aggregates (OPS and OPBC) were prepared with the similar grading of the aggregates with the maximum size of 12 mm. For each concrete mix, the OPBC and OPS aggregates were weighed in dry conditions and immersed in water for 24 hours.

Then they air dried in the laboratory environment for 2-3 hours to obtain an aggregate with an almost saturated surface dry condition. Table 3.2 shows the physical properties of OPBC and OPS aggregates. In addition, Sika ViscoCrete was used as the super-plasticizer (SP), and normal tap water was used as mixing water in this study.

5.2.2 Mix Proportions

To investigate the effectiveness of partial replacement of OPBC with OPS on the properties of OPBC concrete, a grade 55 OPBC concrete was used as control mix. The mix proportions of control concrete with high strength and good workability were achieved by several trial and error methods. The density of this concrete was in the range of semi-lightweight concrete. To reduce the density of this concrete, OPBC coarse aggregates were substituted with OPS coarse aggregates in percentages of 20, 40 and 60 by volume. Mix proportions and slump value of all concrete mixes are shown in Table 5.1.

It was observed that the partial substitution of OPBC by OPS improved the slump value of the mixes. This was mainly due to the shape and surface texture of the aggregates. The OPS aggregate have smooth surfaces which resulted higher workability than the OPBC concrete (Aslam et al., 2016b; Mannan & Ganapathy, 2002). However, all the mixes showed very good workability without any segregation. Mehta and Monteiro (2006) reported that the structural LWAC with the slump value of 50-75 mm is equivalent to 100-125 mm slump of the NWC.

Table 5.1: Details of the concrete mixes (kg/m^3)

Mix code	Cement	Water	w/c	Sand	Coarse aggregate		SP (% cement)	Slump (mm)
					OPBC	OPS		
M-0	480	173	0.36	890	511	0	0.85	85
M-20	480	173	0.36	890	409	72	0.85	95
M-40	480	173	0.36	890	307	144	0.85	90
M-60	480	173	0.36	890	204	216	0.90	100

5.2.3 Test Methods

The slump test was performed before casting of the specimens. The concrete specimens were casted in 100 mm steel cube moulds to determine compressive strength and water absorption. The cylinders of 200 mm height and 100 mm diameter for splitting tensile strength, cylinders of 300 mm height and 150 mm diameter for modulus of elasticity, and prisms of 100 × 100 × 500 mm for flexural strength were used.

The effect of curing conditions on the 28-day compressive strength was investigated under three different curing conditions, 1) continuous moist curing (FW) until the time of testing, 2) 7 days moist curing (7W) in which the specimens were cured in water for 6 days and then in the lab environment, and 3) air-dry curing (AC) in which the specimens did not have any moist curing. The curing water had a temperature of 23 ± 3 °C. The temperature and relative humidity of the lab environment were 31 ± 3 °C and 84 ± 3 %, respectively. At least three test specimens were prepared to obtain average values of mechanical properties at any age.

5.3 Test Results and Discussion

5.3.1 Density

Figure 5.1 shows the relationship between the OPS substitution level and density. Test results show that substitution of OPBC with OPS reduced the density of the concrete. This occurs due to the lower specific gravity of OPS compared to that of OPBC. The density of control OPBC concrete (mix M-0) in both cases of demoulded and dried densities is between 2000-2200 kg/m^3. In this range, the concrete is considered as semi-lightweight (Abouhussien, Hassan, & Ismail, 2015). Substitution of OPBC with OPS in control mix in the substitution levels of 20, 40 and 60% could reduce the dried density about 2.8%, 5.5% and 9.5%, respectively. All the concretes containing OPS aggregates have a dried density less than 2000 kg/m^3. Therefore, they could be considered as lightweight concrete (Aslam, Shafigh, et al., 2016b).

The difference between demoulded and oven dried densities for OPBC concrete mix was 26 kg/m^3. This difference significantly increased with increasing the substitution levels of OPBC with OPS. It was 51, 68 and 138 kg/m^3 for M-20, M-40 and M-60, respectively. The main reason for increasing the difference between these two densities is high water absorption of OPS compared to OPBC.

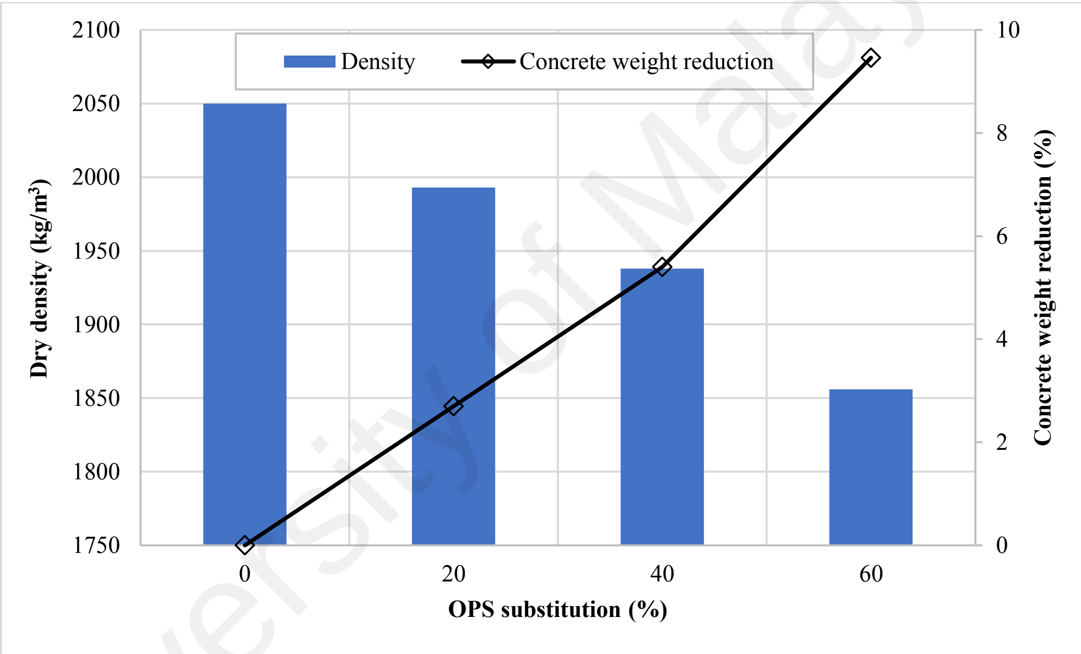


Figure 5.1: Relationship between density, percentage substitution of OPS and concrete weight reduction (%)

Figure 5.2 shows the relationship between dry density and the 28-day compressive strength of concretes with strong correlation. This figure also illustrates the variation of 28-day compressive strength with respect to dry density for other lightweight concrete that were made with different lightweight aggregates. As can be seen in this figure, 28-day compressive strength increases with increasing dry density for all types of lightweight aggregate concrete.

5.3.2 Compressive Strength

The compressive strength development of all concrete mixes under continuous moist curing up to 56 days age is shown in Figure 5.3. The mix M-0 containing OPBC coarse aggregates was considered as the control concrete which showed the 28-day compressive strength of 57 MPa. As can be seen in Figure 5.3, the incorporation of the OPS in OPBC concrete showed very close results of the compressive strength at early ages (1-, 3 days). However, at the age of 7 days the contribution of OPS in the control concrete showed a consistent reduction in the compressive strength. Particularly at later ages of 28- and 56 days. The significant reduction in the compressive strength was observed by the incorporation of OPS in OPBC concrete. The mixes M-20, M-40 and M-60 showed 8.4%, 15.4% and 25.3%, lower 28-day compressive strength compared to control mix (M-0), respectively.

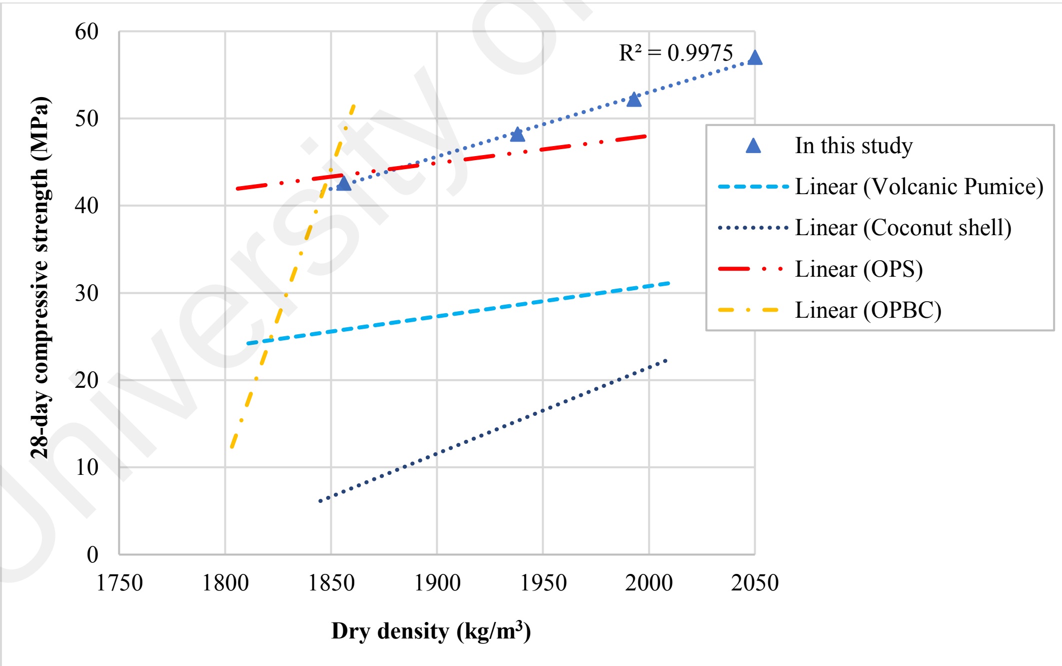


Figure 5.2: The relationship between the dry density and 28-day compressive strength of OPBC-OPS mixes and comparison with OPS concretes (OPSC); palm oil clinker concrete containing OPBC as coarse and fine; high strength lightweight concrete containing scoria aggregate; volcanic pumice lightweight concrete and coconut shell concretes

Source: (Gunasekaran, Kumar, & Lakshmipathy, 2011; Hossain & Khandaker, 2004; Kilic et al., 2003; Mohammed et al., 2014; Mohammed et al., 2013; Shafigh, Jumaat, & Mahmud, 2011; Shafigh, Jumaat, Mahmud, et al., 2011; Shafigh, Mahmud, et al., 2012)

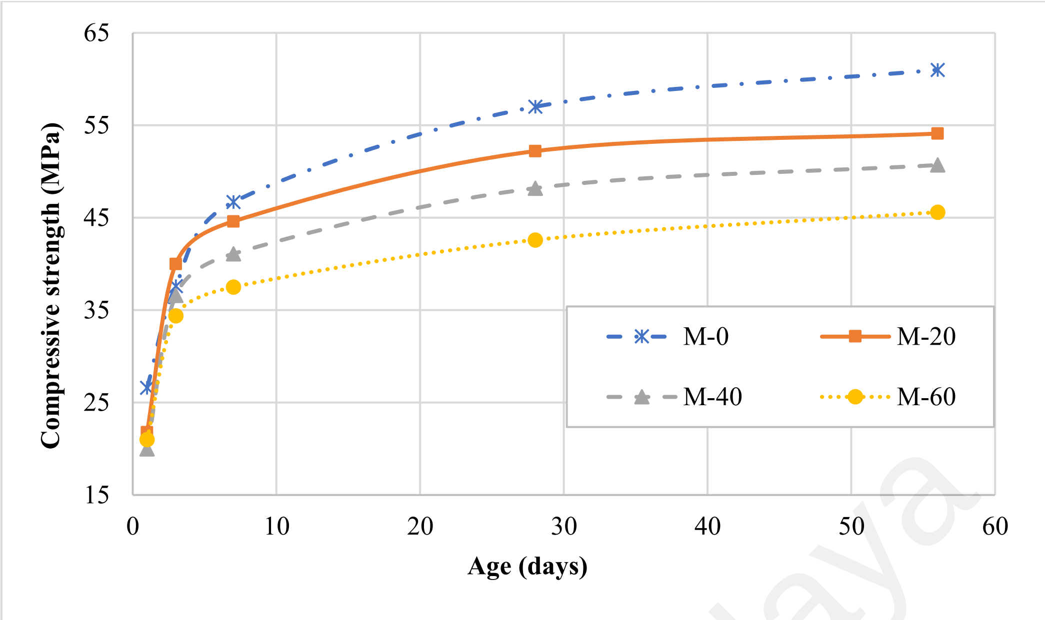


Figure 5.3: Development of compressive strength of the concrete mixes

The reduction of the compressive strength of OPBC-OPS concretes was mostly due to the weak bonding between the OPS aggregates and the cement matrix. Lo et al. (2007) reported that the strength of the LWAC depends on the strength of the utilized aggregates and the cement paste, as well as the interfacial bond between the aggregate and the cement paste. Previous studies (Aslam, Shafigh, et al., 2016a; Mannan et al., 2002; Okpala, 1990) revealed that the failure in OPS concrete is generally due to breakdown of the bond between the cement paste and the aggregates. Weak bond between OPS and cement mortar is mainly due to smooth surface texture on both convex and concave faces of the OPS which sustain lower loads under compression (Mannan et al., 2002). However, it should be noted that the OPBC is an inorganic porous material with rough surface texture (Figure 4.1). Therefore, it is expected that it would have better interfacial lock with the cement paste, and, consequently, showed higher strength. Although, significant reduction on the 28-day compressive strength was observed on OPBC-OPS concretes, it is found that by substituting OPBC with OPS up to 40% the concrete is still in high strength grade.

Results show that the OPBC-OPS concretes achieved 85-88% of the 28-day compressive strength at the 7-day age. While, it was 82% for concrete containing just

OPBC. For an artificial LWAC, the ratio of 7- to 28-day compressive strengths is generally found in the range of 76-87% (Wilson & Malhotra, 1988). The ratio for the 1-, and 3-day to 28-day compressive strength of OPBC-OPS concrete mixes was in the range of 41-49% and 75-80%, respectively.

The relationship between the early ages of 1, 3 and 7 days, and 28 days compressive strengths is shown in Figure 5.4. The prediction models were proposed to compute the 28-day compressive strength from the early ages (1-, 3-, and 7-day). It was observed that the prediction models achieved by 1-, and 3-day results showed low reliability. However, there is strong correlation between 7-day and 28-day compressive strengths. Previous studies also showed that the correlation between early compressive strengths and 28-day compressive strength of lightweight aggregate concretes is strong for 7- and 28-day ages in most types of lightweight aggregates.

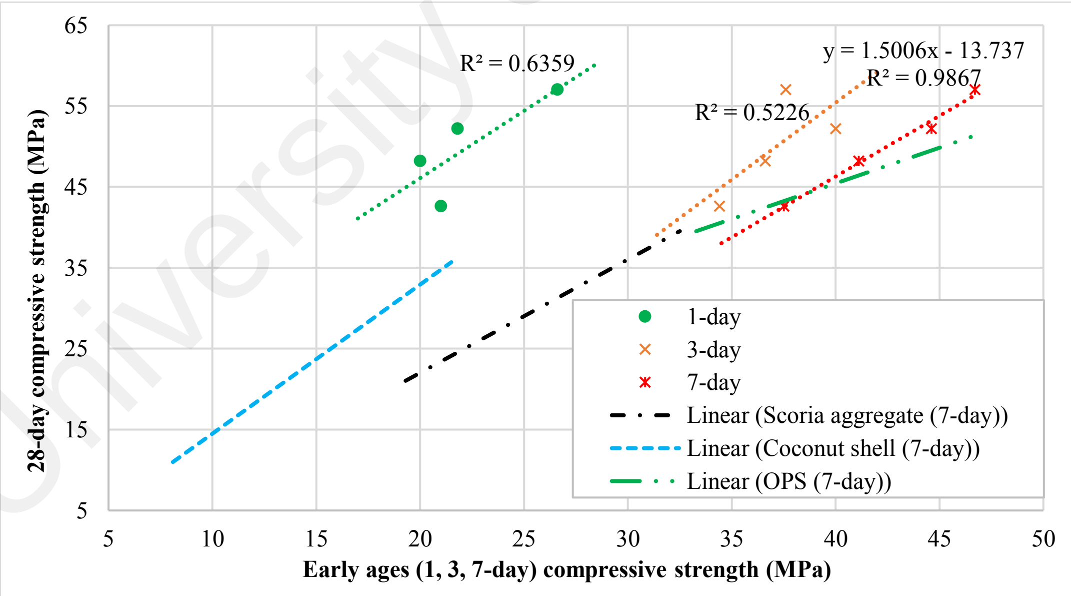


Figure 5.4: The relationship between early ages and 28-day compressive strength of all mixes and comparisons with OPS lightweight concretes; scoria lightweight concrete; and coconut shell lightweight concretes

Source: (Kilic et al., 2003; Olanipekun, Olusola, & Ata, 2006; Shafigh, Jumaat, & Mahmud, 2011)

The comparison of the 28-day compressive strength under continuous moist curing (FW), 7-day water curing (7W) and air-dried (AC) condition of all the mixes is shown in Figure 5.5. As can be seen in this figure, all the concrete mixes showed a reduction in the 28-day compressive strength under AC. Compared to FW, the reduction in the compressive strength of the OPBC concrete mixes by the incorporation of 0, 20, 40 and 60% OPS aggregate were about 4%, 15%, 10% and 3%, respectively. It was observed from the results that the OPBC concrete showed very low reduction in the compressive strength under air-drying condition.

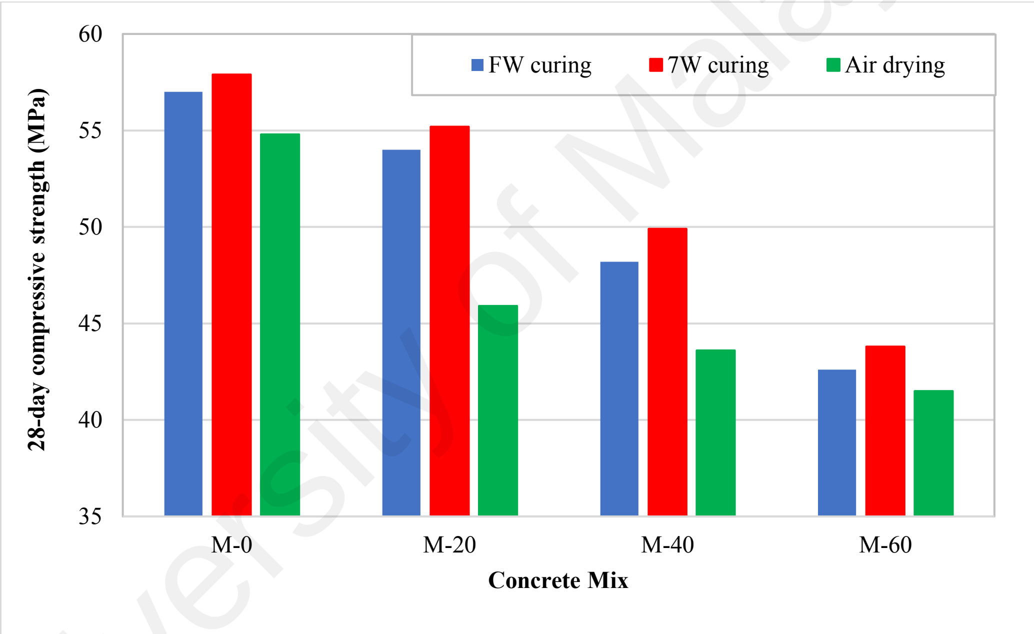


Figure 5.5: 28-day compressive strength with and without curing

As can be seen in Figure 5.5, the partial early curing of 7W showed higher compressive strength for all the mixes. Compared to AC condition, partial early curing of 7W improved the compressive strength of the M-0, M-20, M-40 and M-60 mixes by about 5.4%, 17%, 13% and 5%, respectively. Early water curing therefore caused a greater improvement in compressive strength. However, the compressive strength of all the concrete mixes under 7W curing condition was on average about 2% higher than the compressive strength under FW curing. It was observed that all the mixes under 7W curing condition gave very similar results to that of the continuous moist cured specimens. It seems that due to this reason most of the codes recommended sufficient curing to achieve a maturity equivalent to 7-day moist curing (Haque, 1990).

5.3.3 Stress-strain Relationship of OPBC-OPS Concretes

The stress-strain relationship in compression is one of the important property of concrete to predict the behaviour of structural elements under action loads. From the stress-strain relationships many properties of a concrete can be derived, such as the strains at peak stresses, ultimate strains, fracture toughness, and the modulus of elasticity. The compressive stress-strain relationship is typically divided in four parts; firstly the initial linear stress-strain relationship in which the concrete behaved elastically, secondly the existence of micro-cracks which changes the shape of the curve into a non-linear relationship, thirdly the consequent formation of macro-cracks in which the stress–strain curve bends towards the horizontal and the concrete could not sustain additional loads, and finally the development of larger cracks in that the load carrying capacity of the concrete was reduced and the stress-strain curve descends (Mo, Alengaram, & Jumaat, 2015). The higher strain capacity of concrete shows an ability to absorb movements and also improve the resistance to cracking (Turatsinze & Garros, 2008).

The stress-strain relationship for the concrete mixes of M-0, M-40 and M-60 is shown in Figure 5.6. The strain at the peak stress (ε_0) of these mixes was found to be 0.00214, 0.00278 and 0.00418, respectively. The incorporation of 40 and 60% OPS aggregate in OPBC concrete showed 23% and 49%, higher strain (ε_0) values compared to control OPBC concrete, respectively. Shafigh et al. (2012) reported that the volume of OPS aggregates in the concrete mixtures significantly affects the peak and rupture strain values. The ε_0 for the NWC is normally in the range of 0.0015 to 0.002 (Akbar, 2008). Results of this study showed that ε_0 of OPBC concrete is similar to conventional aggregate concrete while incorporation of OPS in OPBC concrete increased the ε_0 values. The mix M-60 showed the higher peak stress which may be resulted higher number of cracks compared to all other concretes, especially, the control OPBC concrete. It was observed that as the amount of OPS aggregate increases, the number of cracks also increases which in turn may reduce the brittle nature of the control OPBC concrete. This could be more advantageous to overcome the shrinkage cracks on the structural component. Balendran, Zhou, Nadeem, and Leung (2002) reported that compared to NWC, the lightweight aggregate concrete is more inelastic in nature which causes a sudden failure. This fact was observed in stress-strain curve of control OPBC concrete. However, OPBC-OPS concretes showed ductile behaviour. These results indicated that the type of lightweight aggregate and their properties has significant effect on the stress-strain behaviour of a concrete.

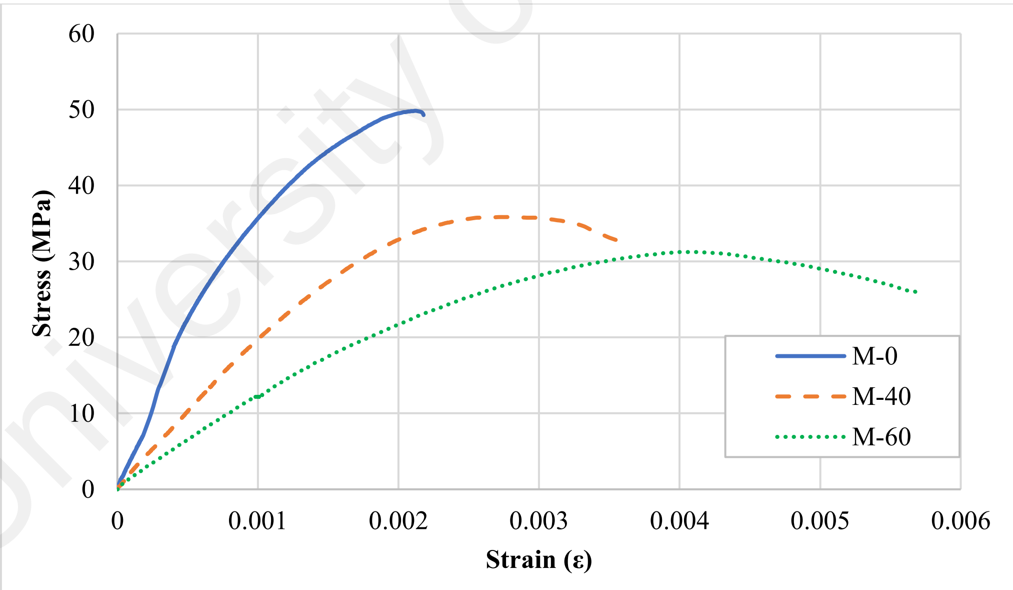


Figure 5.6: Stress-strain curves for the prepared concrete mixes

The experimental results of the strains (ε_0) at the peak stress were also computed from the compressive strengths by using several prediction models proposed by different researchers, as can be seen in Figure 5.7. The details of the prediction models for the strains from peak stress are summarized in Table 5.2.

Table 5.2: Models to predict the strain at peak stress for concrete

Eq. No.	Equations	Remarks	References
5.1	$\varepsilon_0 = (70\, f_{cu}^{0.44} - 15) * 10^{-5}$	To predict the strains at peak stress from 28-day compressive strength	(Tasnimi, 2004)
5.2	$\varepsilon_0 = 0.000875\, f_{cu}^{0.25}$	To predict the strains at peak stress by stress-strain relationship	(Popovics, 1970)
5.3	$\varepsilon_0 = (0.398 f_{cu} + 18.147) * 10^{-4}$	To predict strain at peak stress for normal and lightweight concretes	(Almusallam & Alsayed, 1995)
5.4	$\varepsilon_0 = (7.1 f_{cu} + 1680) * 10^{-6}$	To predict strain at peak stress for the NWC	(Carreira & Chu, 1985)
5.5	$\varepsilon_0 = (2 f_{cu}) * 10^{-5} + 0.0008$	To predict strain at peak stress for the NWC	(Hussin, Zhuge, Bullen, & Lokuge, 2013)
5.6	$\varepsilon_0 = (0.0546 + 0.003713 f_{cu}) * 10^{-2}$	To predict strain at peak stress for the NWC	(Ros, 1950)
5.7	$\varepsilon_0 = (1.6 + 0.01 f_{cu}) * 10^{-3}$	To predict strain at peak stress for the NWC	(Tadros, 1970)
5.8	$\varepsilon_0 = (1.65 f_{cu}) * 10^{-5} + 0.0008$	To predict the strains at peak stress for high strength concrete	(Ahmad & Shah, 1985)
5.9	$\varepsilon_0 = (1.49 f_{cu}) * 10^{-5} + 0.00195$	To predict the strains at peak stress	(Fafitis & Shah, 1985)
5.10	$\varepsilon_0 = (3.47 f_{cu}^{0.25})(31.5 - 3.47 f_{cu}^{0.25}) * 10^{-5}$	To predict the strains of concretes with the wide range of compressive strengths	(Desayi & Krishnan, 1964)
5.11	$\varepsilon_0 = \left[\left(0.626 \frac{f_{cu}}{f^*} - 4.33\right) * 10^{-7}\right]^{0.5} + 0.00076$	To predict the strains of concretes with the wide range of compressive strengths (10 to 100 MPa)	(De Nicolo, Pani, & Pozzo, 1994)

where, ε_0 is the strain at peak stress, f_{cu} is the 28-day compressive strength (MPa), and $f^*$is compressive strength equal to 1 MPa. As can be seen in Figure 5.7, most of the equations failed to obtain good agreement with the experimental data of the OPBC-OPS lightweight concretes. However, some equations are found to successfully predict the strains at peak stress for the concrete mix containing only OPBC. This may be due to the ductile behaviour of OPS aggregates compared to brittle nature of OPBC. Therefore, a new prediction model is described in Equation 5.12 to estimate ε_0 of concretes containing OPS is required. The experimental data presented in this study suggests that we approximate the ε_0 using the following equation with R-squared of 96%.

$$\varepsilon_0 = 18.938\ f_{cu}^{-2.256} \tag{5.12}$$

where, ε_0 is the peak strain and the f_{cu} is the 28-day compressive strength (MPa) of the concrete mixes.

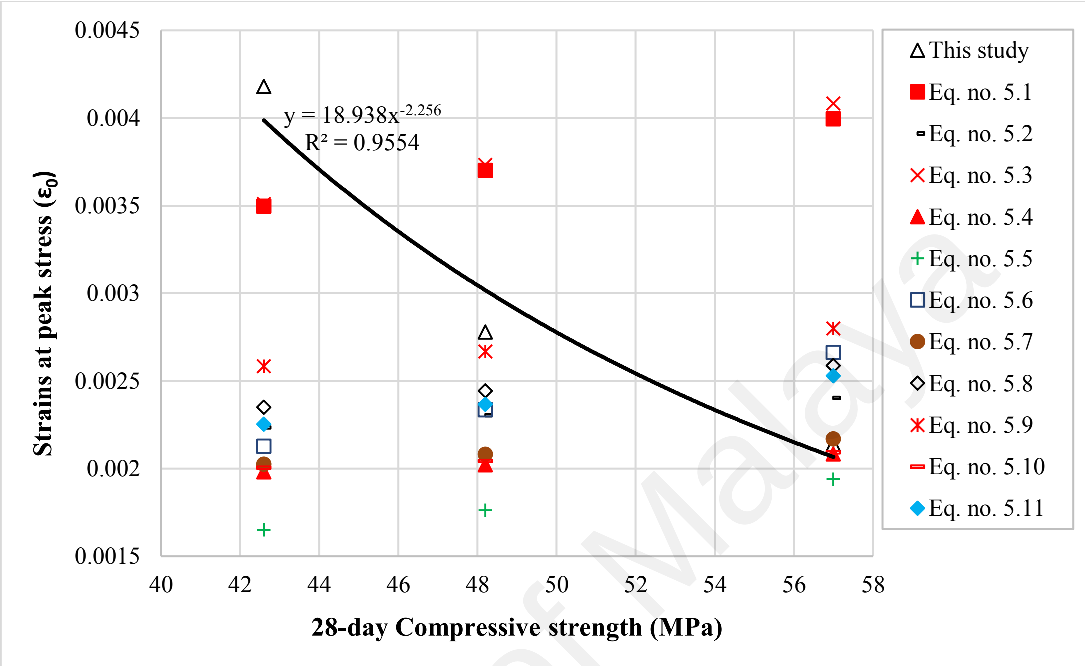


Figure 5.7: Relationship between the 28-day compressive strength and strains at peak stress and comparison with predicted values

5.3.4 Splitting Tensile Strength

In real life applications, the engineers and researchers working with reinforced concrete normally ignore the concrete with low tensile strength and provide steel reinforcement to carry the tensile loading. However, in some structures, such as dams, design of highway and airfield slabs, concrete pavements, shear strength and other slabs, it is impractical to use steel reinforcement. Therefore, a reliable value of the splitting tensile strength of concrete is very necessary, especially to judge the safety of the dams under seismic loading (Mehta & Monteiro, 2006; Neville & Brooks, 2008). Bhanja and Sengupta (2005) reported that due to low tensile strength of concrete, it easily produces cracks in tension region which may cause durability and serviceability problems.

The relationship between the splitting tensile and the OPS substitution level at 3, 7 and 28-day ages are shown in Figure 5.8. Test results showed that all the concrete mixes have a splitting tensile strength more than 2.0 MPa from 3-day age. A 28-day splitting tensile strength of 2.0 MPa is minimum requirement for structural lightweight aggregate concrete (Kockal & Ozturan, 2011). It is found that, the replacement of OPBC with OPS resulted a reduction in the splitting tensile strength at all ages. The slope of lines is found to increase by time in Figure 5.8. This shows that although the splitting tensile strength increased by time in all concrete mixes, however, the improvement was reduced as OPS content increased. The ratios of the 3-, and 7-day splitting tensile to the 28-day splitting tensile strength of the OPBC-OPS mixes was found in the ranges of 74-80% and 79-88%, respectively.

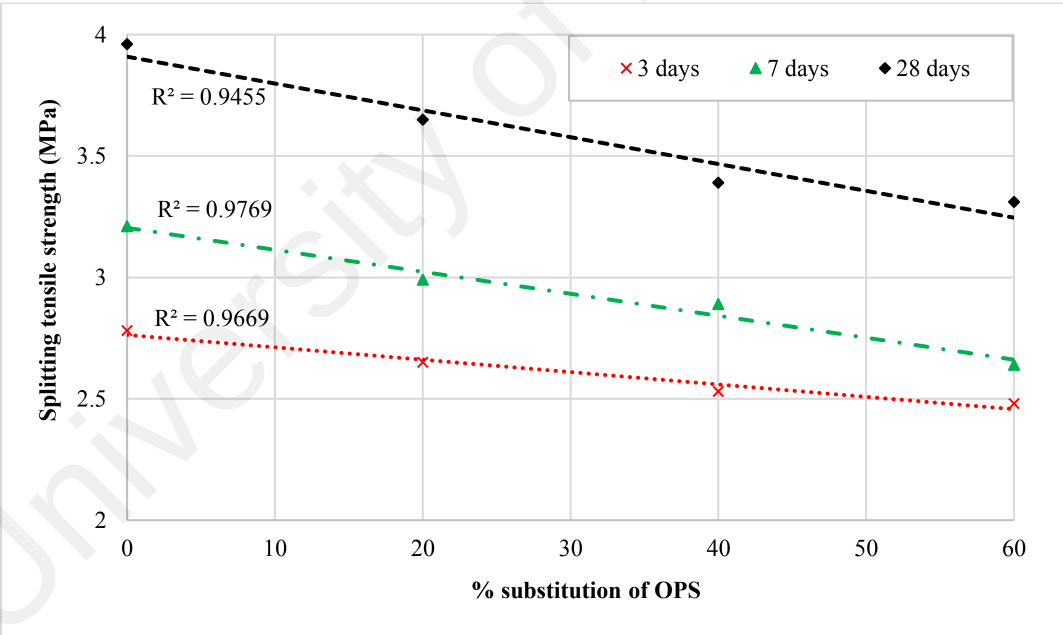


Figure 5.8: Relationship between the splitting tensile strength and the substitution of OPS

In this study, the splitting tensile to the compressive strength ratio for OPBC concrete and OPBC-OPS concretes was about 6.9% and 7-7.8%, respectively. Generally, this ratio for NWC falls in the range of 8-14% Wilson & Kosmatka (2011), however, compared to NWC, the tensile strength to compressive strength ratio is lower for LWAC of equivalent grade (Haque et al., 2004). Additionally, it was reported that for high strength LWC, in continuous moist curing; the splitting tensile strength is generally 6-7% of the compressive strength (Holm & Bremner, 2000). Reports show that this ratio for OPS lightweight concrete is 8-10% (Shafigh et al., 2012) and for OPBC concrete is 5.7-8.9% (Mohammed et al., 2013; 2014).

The experimental results of the splitting tensile strength were also compared with the predicted results as shown in Figure 5.9. The Eq. (5.13) was proposed by ACI318 (2005) for NWC with the 28-day cylinder compressive strength range of 21-83 MPa. Whereas, the Eq. (5.14) was proposed for LWAC containing cold-bonded fly ash with a cubical compressive strength range of 20-47 MPa (Gesoglu et al., 2004), and Eq. (5.15) was proposed from CEB-FIP (2010).

$$f_t = 0.59\,(f_{cy})^{0.5} \tag{5.13}$$

$$f_t = 0.27\,\sqrt[3]{f_{cu}^2} \tag{5.14}$$

$$f_t = 0.301\,(f_{cy})^{0.67} \tag{5.15}$$

where, f_t is the splitting tensile strength (MPa), f_{cu} and f_{cy} are the cube and cylindrical compressive strengths (MPa), respectively.

It was observed that the equations (5.14) and (5.15) can predict very close results to the experimental values with an error of about 6%. While this error was about 15% for the equation (5.13).

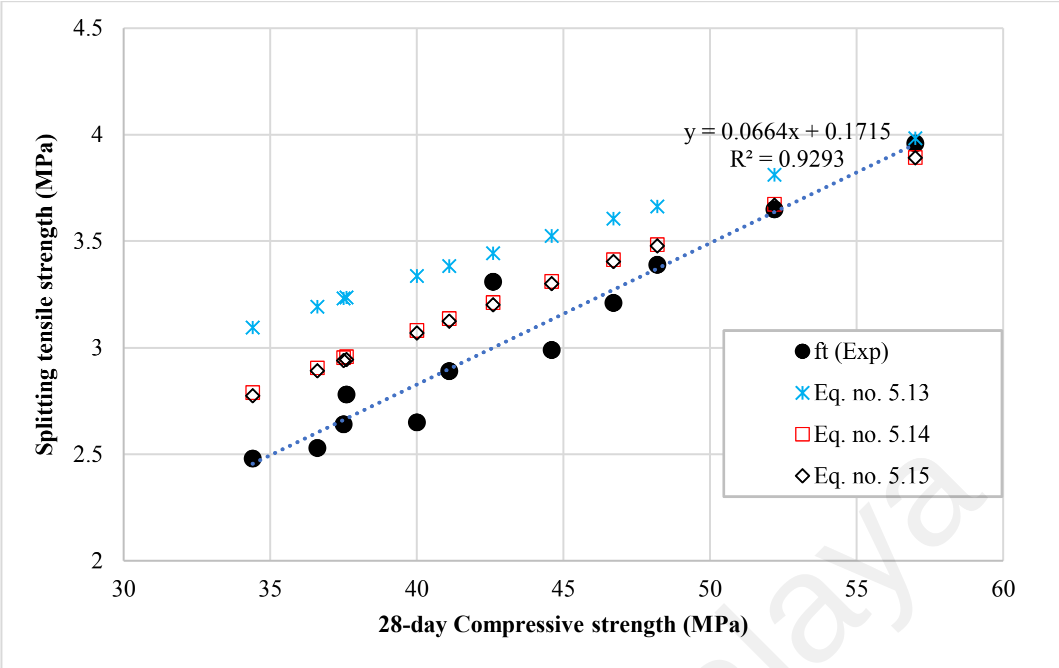


Figure 5.9: Experimental and predicted splitting tensile strength of concretes

5.3.5 Flexural Strength

The effect of the flexural strength of all the mixes at 7 and 28-days are shown in Figure 5.10. It can be seen that the contribution of OPS in the OPBC concrete reduced the flexural strength. Compared to control concrete, the reduction of the 28-day flexural strength was about 15.3%, 24.5% and 31% at the OPS substitution levels of 20, 40 and 60%, respectively. The flexural strength of the NWC with a 28-day compressive strength of about 34-55 MPa found in the range of 5-6 MPa (Mehta & Monteiro, 2006), whereas this range for OPS lightweight concrete was found in the range of 2.1-4.9 MPa (Alengaram et al., 2008; Okpala, 1990; Teo et al., 2006). With the highest contribution of OPS in OPBC concrete (mix M-60) the 28-day flexural strength was about 5.3 MPa, which is still in the acceptable range of the NWC.

The control OPBC concrete showed 26% higher 28-day flexural strength compared to its 7-day strength. Whereas, this increment in OPBC-OPS mixes was about 20% on average. On the other hand, the 7-day compressive strength of control OPBC concrete and OPBC-OPS concretes were about 82% and 86% of the 28-day compressive strength, respectively. As can be seen in Figure 5.10, the 7-day splitting tensile strength of the prepared concretes was found in the range of 80-85% of the 28-day splitting tensile strength. The splitting tensile to flexural strength ratio for the control (M-0) mix is about 51%, whereas, this ratio for OPBC-OPS mixes was found in the range of 56-63%. Shafigh et al. (2012b) reported that for a wide range of compressive strength of OPS concretes (up to 53 MPa), the splitting to flexural strength ratio varies in the range of 51 to 72%, and this ratio is affected by the amount of OPS in the mixture. Therefore, a structural member made of these lightweight aggregate concretes can be oppressed at an early age of 7 days.

The flexural to compressive strength ratio for OPBC concrete was about 13%. This ratio for OPBC-OPS concretes, however, was found in the range of 12-12.5%. While, this ratio for NWC and high strength lightweight aggregate concrete was found in the ranges of 11.6-13.5% (Holm & Bremner, 2000) and 9-11% (Mehta & Monteiro, 2006), respectively. It can therefore be concluded that all the OPBC-OPS concretes had a similar flexural strength, and flexural to compressive strength ratio to the NWC and high strength lightweight aggregate concrete.

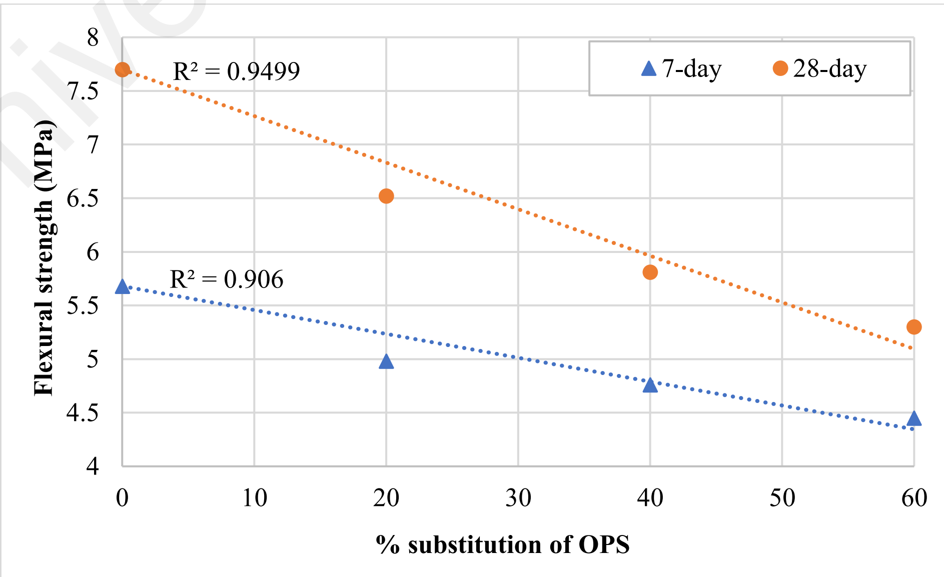


Figure 5.10: Relationship between the flexural strength and the substitution of OPS

The relationship between the experimental and predicted flexural strength results and the corresponding compressive strengths is shown in Figure 5.11. The experimental test results were also compared with estimated values from proposed equations. The Eq. (5.16) was proposed for crushed OPS concrete with cube compressive strength range of 35-53 MPa (Shafigh et al., 2012). CEB-FIP (1977) proposed Eq. (5.17) for expanded shake and clay LWAC with cube compressive strength range of 20-60 MPa, and Eq. (5.18) was proposed for the flexural strength of the high strength LWC (Zhang & Gjvorv, 1991).

$$f_r = 0.12 f_{cu}^{1.03} \quad (5.16)$$

$$f_r = 0.46 \sqrt[3]{f_{cu}^2} \quad (5.17)$$

$$f_r = 0.73 \sqrt{f_{cu}} \quad (5.18)$$

where, f_r is the flexural strength (MPa), and f_{cu} is the cube compressive strength (MPa) of the concrete.

It was observed that the equations (5.16) and (5.17) predicted very close results to the experimental values with the average difference of about 7%. However, the equation (5.18) showed good correlation between the experimental and predicted values for the concretes (mixes M-40 and M-60) with the 28-day compressive strength up to 45 MPa, whereas, for the grade 50 and above concretes (mixes M-20 and M-0) this equation underestimated the predicted results.

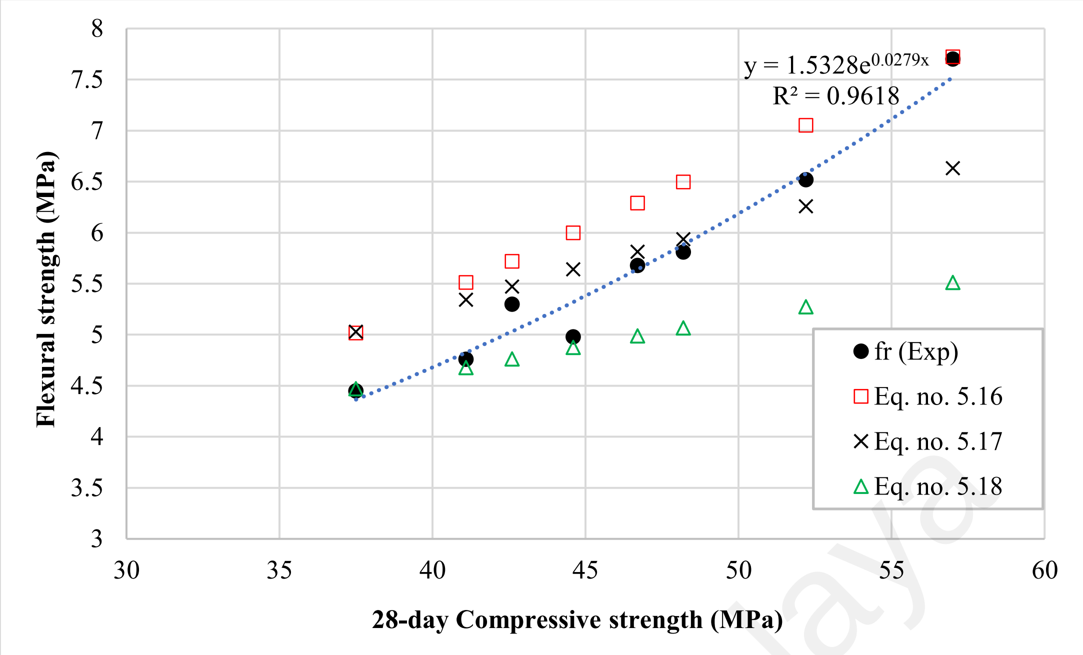


Figure 5.11: Experimental and predicted flexural strength results of the concretes

5.3.6 Modulus of Elasticity

The modulus of elasticity of the M-0, M-20, M-40 and M-60 mixes was 28.2, 23.4, 20.0 and 13.0 GPa, respectively. Similar to the other mechanical properties, the elastic modulus was also reduced by increasing OPS in OPBC concrete. The reduction of the modulus of elasticity at 20%, 40% and 60% substitution levels was about 17%, 29% and 54%, respectively.

The volume fraction, density and the modulus of elasticity of the aggregates and the characterization of the interfacial transition zone plays very important role to determine the elastic behaviour of the concrete. Among the aggregate characteristics, the porosity seems to be most important, because it determines the stiffness and also controls the matrix strains. In general, the modulus of elasticity of LWAC ranges from 10 to 24 GPa, which is about 50 - 75 % of the elastic modulus of NWC of the same grade (Holm & Bremner, 2000).

In this study, the main difference in all the concrete mixes was the type and volume of coarse aggregates. As the incorporation of OPS aggregate in OPBC concrete increased, the modulus of elasticity was reduced. This might be due to lower modulus of elasticity of OPS compared to OPBC as well as weakness in the interfacial transition zone between OPS and cement matrix due to smooth surface texture of OPS in both concave and convex surfaces.

Although the modulus of elasticity for 60% substitution (mix M60) is significantly less than control OPBC concrete, however, it is still in the normal range for LWACs. The low modulus of elasticity may cause excessive deflection in flexural reinforced concrete members. However, it may be beneficial for reducing the internal stress concentrations in concretes and consequently reducing the micro-cracks (Bremner & Holm, 1995).

The modulus of elasticity is related to the compressive strength of concrete. As the compressive strength increased the modulus of elasticity also increases (CEB/FIP, 1977). Figure 5.12 illustrates the relationship between the 28-day compressive strength and the modulus of elasticity of all concrete mixes. A strong linear correlation with R^2 of 0.99 was achieved between the 28-day compressive strength and the modulus of elasticity. Whereas, Figure 5.13 shows the variation of the modulus of elasticity with the contribution of OPS aggregate in the OPBC concrete. The strong linear correlation with R^2 of 0.98 was achieved.

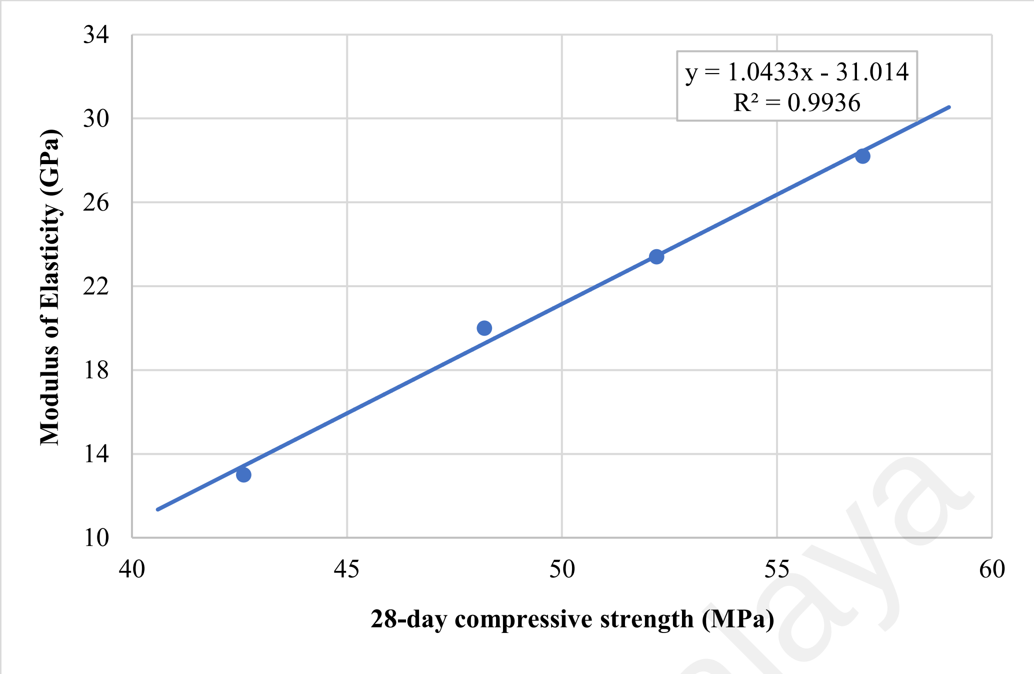


Figure 5.12: Relationship between the modulus of elasticity and 28-day compressive strength

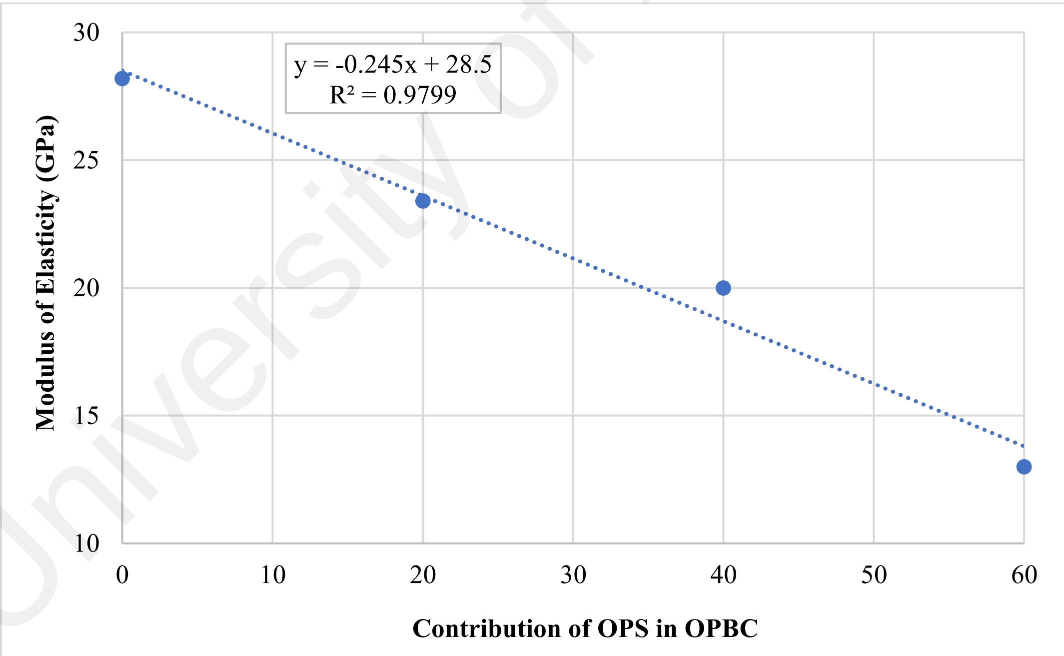


Figure 5.13: Relationship between the modulus of elasticity and contribution of OPS in OPBC

Several prediction equations were also used to compute the modulus of elasticity of the prepared concrete mixes, the detailed description of the prediction models and the predicted E-values are mentioned in Table 5.3 and Table 5.4, respectively.

Table 5.3: Models to predict the modulus of elasticity

Eq. No.	Equations	Remarks	References
5.19	$E = 0.03w^{1.5} f_{cy}^{0.5}$	To predict the E-value of lightweight pumice concrete with a 28-day cylinder compressive strength of 16-35 MPa and density of about 1460-2185 kg/m^3.	(Hossain et al., 2011)
5.20	$E = 5f_{cu}^{0.33}(w/2400)^2$	To predict the E-value of the OPS concrete with 28-day cube compressive strength of 25-39 MPa and air-dry density of 1640-1890 kg/m^3.	(Alengaram et al., 2011)
5.21	$E = 2.168f_{cy}^{0.535}$	To predict E-value for artificial LWA concretes with a cylindrical compressive strength of about 15-55 MPa.	(Tasnimi, 2004)
5.22	$E = 9.1(w/2400)^2 f_{cu}^2$	To predict the modulus of elasticity of LWACs.	(Short, 1978)
5.23	$E = 0.0017w^2 f_{cu}^{0.33}$	To predict the E value of the concretes.	(BS-8110, 1986)
5.24	$E = (0.062+0.0297f_{cy}^{0.5})\ w^{1.5}$	To predict the E-value of high strength lightweight concrete.	(Slate et al., 1986)

where, E is the modulus of elasticity (GPa), w is the dry density (kg/m^3), f_{cy} and f_{cu} are the cylinder and cube compressive strengths (MPa).

Table 5.4: Measured and predicted modulus of elasticity

Mix code	28-day compressive strength (MPa)	Dry density (kg/m^3)	Measured modulus of elasticity (GPa)	Estimated modulus of elasticity (GPa)					
				Eq. (5.19)	Eq. (5.20)	Eq. (5.21)	Eq. (5.22)	Eq. (5.23)	Eq. (5.24)
M-0	57	2050	28.2	18.8	13.9	16.7	21.6	27.2	18.6
M-20	52.2	1993	23.4	17.3	12.7	15.9	17.0	24.9	17.0
M-40	48.2	1938	20.0	15.9	11.7	15.3	13.8	22.9	15.7
M-60	42.6	1856	13.0	14.0	10.3	14.3	9.9	20.2	13.8

5.3.7 Water Absorption

Water absorption is an essential factor which is primarily related to the porosity and the durability of the concrete. The effect of the replacement of OPBC with OPS aggregate on moderate (24 hours) and final (72 hours) water absorption of the mixes was measured at the age of 56 days, as shown in Figure 5.14. It was observed that the water absorption of the mixes containing OPS aggregates was higher than that of the control OPBC concrete. It was consistently increased when the amount of OPS aggregate in the mixture was increased. This was mainly due to the high-water absorption of the OPS aggregates, as can be seen in Table 4.1. It was reported by Topcu and Uygunoglu (2010) that the replacement of conventional aggregates with natural LWAs, such as tuff, diatomite and pumice increased water absorption of the concrete. Teo et al. (2007) reported that regardless the type of curing, the water absorption of OPS concrete is more than 10%. Neville (2008) reported that although concrete quality cannot be predicted by water absorption, however, good concretes generally have a rate of absorption less than 10% by mass. On the other hand, Kosmatka et al. (2002) stated that the concrete with high quality usually has water absorption less than 5%. In this study, it was observed that although the incorporation of OPS in OPBC concrete increased the absorption of the concretes, the highest contribution of OPS showed the overall absorption less than 10%. However, based on Figure 5.14, the final water absorption of OPBC-OPS concrete would be less than 5% if OPS contribution in OPBC concrete is less than 40%.

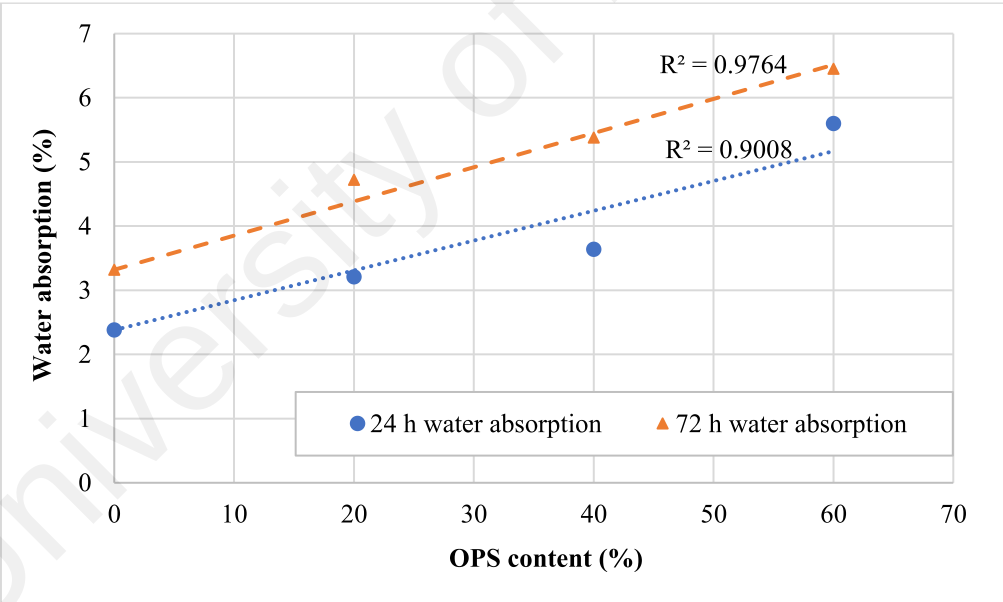


Figure 5.14: Relationship between water absorption and the OPS content

5.4 Conclusions

In this study, a semi-lightweight aggregate concrete was made of coarse oil-palm-boiler clinker (OPBC) was partially substituted with coarse oil palm shell (OPS) aggregate to investigate possibility of producing OPBC high-strength lightweight concrete. The effectiveness of this substitution on density, mechanical properties and water absorption of the concretes was investigated. Based on the test results, the following conclusions can be drawn:

1. The density of concrete is found to decrease with the substitution of the OPBC with OPS. The substitution between 20 to 40 % could turn high-strength semi-lightweight OPBC concrete into high-strength lightweight concrete.
2. All concretes containing OPBC and OPS lightweight aggregates showed lower compressive strength under air drying. However, six days moist curing after demoulding is enough to achieve equivalent compressive strength under full water curing.
3. Contribution of OPS in OPBC concrete up to 60% could significantly increase the strain at the peak stress (ε_0) of the concrete mixes. This shows that OPS lightweight aggregate can improve ductility performance of lightweight aggregate concretes.
4. Although, OPBC concretes containing OPS had lower tensile strengths compared to OPBC concrete, the splitting tensile strength was found to be in the usual range for structural lightweight aggregate concrete. Moreover, the OPBC-OPS concretes showed flexural strength and flexural/compressive strength ratio similar to NWC and high strength lightweight aggregate concretes.
5. The modulus of elasticity of the OPBC concrete was reduced by the incorporation of the OPS aggregates. The reduction of the modulus of elasticity at high volume substitution of 60% was found to be very significant.
6. Water absorption of the OPBC concrete increased linearly by increasing OPS content in this concrete. This occurs due to higher water absorption of OPS compared to OPBC. The results of the current study suggest that the substitution level should be limited to 20-40% (by volume).

CHAPTER 6: DRYING SHRINKAGE BEHAVIOUR OF STRUCTURAL LIGHTWEIGHT AGGREGATE CONCRETE CONTAINING BLENDED OIL-PALM BIO-PRODUCTS

6.1 Introduction

The increase in shrinkage increases proportionally with the age of concrete. It shrinks when it is exposed to a drying environment which causes an increase in tensile stress leading to cracking and external deflection, before the concrete is subjected to loading (Day, 2003). The shrinkage of a concrete is influenced by the amount of mixing, time after addition of water, temperature fluctuation, placement, and curing (McKeen & Ledbetter, 1969). The higher the amount of water present in the fresh concrete the highly drying shrinkage will be affected (Day, 2003). The makeup of concrete is very important because each ingredient have distinctive characteristics which contribute to the shrinkage of concrete. Drying shrinkage can occur in beams, slabs, columns, foundations and causes stress loss in prestressed members and failure of joints (Shafigh, Alengaram, et al., 2013).

In the case of lightweight aggregate concrete (LWAC), the drying shrinkage is greater than conventional concrete and is mostly affected by the properties and the amount of aggregates (Satish & Berntsson, 2003). The fact is that the shrinkage of concrete is essentially governed by the cement paste due to its contraction. Because the water inside C-S-H is removed in the drying condition. Wongkeo, Thongsanitgarn, and Chaipanich (2012) reported that when small capillary pores (pores less than 50 nm) losses the water, it significantly affects the change in volume of concrete. The degree of hydration or change in volume of concrete affects the drying shrinkage strain of concrete (Basma & Jawad, 1995). One of the main source to oppose this contraction is by the addition of the aggregates to restrain the shrinkage of Portland cement paste to a point that the concrete becomes a practical material (CEB/FIP, 1977). Since structural LWAC is modified at these two factors, it is expected that its behaviour differs from that observed in normal weight concrete (NWC). Firstly, the use of less rigid porous aggregates decreases the restriction effect on paste deformation (Zhang, Li, & Paramasivam, 2005). Secondly, either for strength purposes or for reasons of workability and stability of the mixtures, lightweight concrete (LWC) is usually characterized by larger volumes of better quality paste and lower volume of coarse aggregates (Bogas, 2011; Holm & Bremner, 2000). Therefore, the long-term shrinkage of lightweight concrete should be higher than that of NWC of the same grade. On the other hand, it should be noted that water absorbed by lightweight aggregate (LWA) is released later into the paste due to internal curing, which compensates for the initial water lost by drying and self-desiccation. Because of the internal curing and the continued hydration of the paste, the deformation resistance of the matrix is higher and therefore less water is available for evaporation (Selih & Bremner, 1996). The combination of all these affecting factors together with different variability of lightweight aggregates, the higher shrinkage in LWAC compared to NWC was reported (Coquillat, 1986; Hossain & Lachemi, 2007; Shafigh, Ghafari, Mahmud, & Jumaat, 2014). However, some reports showed that shrinkage in LWAC is less than or similar to shrinkage of NWC (Shafigh, Ghafari, et al., 2014; Wegen & Bijen, 1985). For general design work, it was suggested that the shrinkage of LWAC is between 1.4 and 2 times that of NWC (Clarke, 2002). The drying shrinkage values for structural lightweight concrete may vary between 0.04% and 0.15% (Lamond, 2006).

Since last two decades, oil palm shell (OPS) or palm kernel shell (PKS) has been used as a LWA for producing structural LWAC with a density of 20-25% lower than NWC (Shafigh et al., 2012). There is a little information in the literature concerning the shrinkage of OPS concrete. The drying shrinkage of OPS concrete was first investigated by Abdullah (1996), who reported that OPS concrete showed about 5 times higher drying shrinkage than the NWC. Mannan and Ganapathy (2002) investigated the drying shrinkage of OPS concrete and NWC up to 90 days age. They reported that the drying shrinkage of both the concretes was increased with age but OPS concrete showed higher increment. The OPS concrete showed 14% higher drying shrinkage compared to NWC at the age of 90 days. Alengaram (2009) measured the drying shrinkage of several types of OPS lightweight concrete without any initial moist curing. The specimens were exposed to laboratory environment immediately after demoulding with average humidity of 75% and temperature of 30 ^oC. The cement content, total binder content (cement + fly ash + silica fume), water to binder ratio and 28-day compressive strength varied between 500 and 560 kg/m^3, 560-595 kg/m^3, 0.30-0.35 and 22-38 MPa, respectively. He reported that drying shrinkage of OPS concrete at 28, 56 and 90 days is in the range of 160-520, 300-990 and 540-1300 micro-strain, respectively. He reported that structural lightweight OPS concrete has high drying shrinkage due to high cement and OPS (as coarse aggregate) contents.

The drying shrinkage of OPS concretes with the substitution of fly ash with cement and normal sand with oil-palm-boiler clinker (OPBC) sand was studied by (Shafigh, Alengaram, et al., 2013; Shafigh, Mahmud, et al., 2014). They reported that the use of 10% fly ash in OPS high strength concrete did not affect the drying shrinkage of the concrete. However, generally, for higher percentage replacement levels of 30% and 50%, the drying shrinkage increased but was not significant. The substitution of normal sand with OPBC sand in OPS concrete does not influence the drying shrinkage (Shafigh et al., 2014b). However, they further conducted a comparative study of OPS and expanded clay lightweight concretes (Shafigh et al., 2014). The investigations showed that, although the OPS concrete had a better engineering properties and greater efficiency factor (compressive strength to density ratio) but it showed twice the amount of drying shrinkage at early ages. However, this ratio reduces significantly at later ages. Recently, Mo et al. (2016) investigated the durability properties of a sustainable concrete by using OPS as coarse and manufactured sand as fine aggregates. The GGBFS was used as partial cement replacement at 20, 40 and 60% levels in the OPS concrete. They reported the drying shrinkage values of about 740-760 micro-strain at GGBFS replacement levels of 20% and 40%, it was closer to that of the control OPS concrete. However, the substitution of 60% GGBFS increased the shrinkage values by about 20% compared to control OPS concrete.

Previous studies (Mannan et al., 2006; Teo et al., 2007) revealed that OPS concrete has good mechanical properties and durability performance. However, this concrete has some drawbacks which needs to be solved before it is used in practice. One of the drawbacks is its high drying shrinkage compared to artificial lightweight concretes and also normal weight concrete. The main objective of this study is to resolve the high drying shrinkage problem of OPS concrete. Shafigh et al. (2013) reported that the main reason for high drying shrinkage of OPS concrete is due to high cement content and OPS (as coarse) contents. Therefore, it seems that one way to reduce the drying shrinkage of this concrete is to reduce the volume of coarse OPS in concrete mixture. For this purpose, oil-palm-boiler clinker (OPBC) as lightweight aggregate was selected to be used as partially replacement with OPS. The reason for choosing the incorporation of OPBC aggregates in OPS concrete is that OPBC is a solid waste and lightweight material produced from burning of solid wastes in the boiler combustion process in palm oil mills. It is like a porous stone, grey in colour, flaky and irregular in shape (Ahmad et al., 2007). Previous studies (Abdullahi et al., 2008; Chan & Robani, 2008; Zakaria, 1986) have shown that OPBC can be used as coarse lightweight aggregate in concrete. The density and the 28-day compressive strength of OPBC concrete fulfil the requirements of structural LWAC. Therefore, in this study, two types of waste originated from the palm oil industry, namely OPS and OPBC, were used as coarse aggregates. The optimum substitution level of OPBC in OPS concrete to reduce drying shrinkage of OPS concrete was identified.

6.2 Experimental Program

6.2.1 Materials Used

Ordinary Portland cement (OPC), having 3-, 7- and 28-day compressive strength of 26, 36 and 48 MPa, respectively was used as binder conforming to MS522, part-1:2003. The specific gravity and Blaine specific surface area of the cement were 3.14 and 3510 cm^2/g, respectively. A Sika Viscocrete containing total chloride ion content of ≤0.1 and compatible with all type of cements according to BS-5075 was used as super-plasticizer (SP). The local mining sand with a fineness modulus of 2.89, specific gravity of 2.68 and maximum grain size of 4.75 mm was used as fine aggregate. The OPS and OPBC were collected from a local palm oil mill. They were used as coarse aggregate. The OPS aggregates were placed in open air for 6 months to remove fibres from the surface and then washed (Shafigh, Jumaat, & Mahmud, 2011). While the OPBC aggregates were crushed using a crushing machine. After washing and crushing, both aggregates were sieved to get the same grading of the aggregates (Figure 6.1). The physical and mechanical properties of aggregates are shown in Table 6.1. From Table 6.1, it can be seen that OPBC has higher density and lower water absorption compared to OPS. However, crushing, impact and abrasion values of the coarse aggregates show that OPBC is weaker in strength compared to OPS. As can be seen in Figure 6.2, an OPBC grain is round in shape and has many porosities on the surface while OPS is flaky and elongated with no porosity on the surface.

Table 6.1: Physical and mechanical properties of aggregates

Physical and mechanical properties	Coarse aggregate		Fine aggregate
	OPS	OPBC	Normal Sand
Specific gravity	1.19	1.69	2.68
Compacted bulk density (kg/m^3)	610	860	1657
24 h water absorption (%)	20.5	7.0	1.2
Crushing value (%)	0.2	21.2	-
Impact value (%)	5.5	36.3	-
Abrasion value (%)	5.7	23.9	-

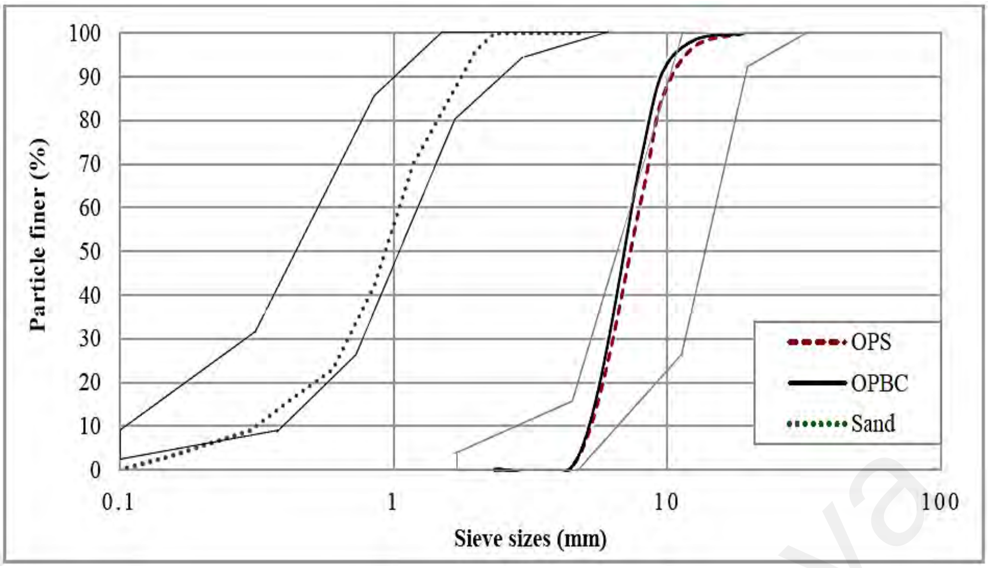


Figure 6.1: The grading of the aggregates as per limits specified by ASTM C-33

Figure 6.2: Oil palm shell (left) and oil-palm-boiler clinker (right)

6.2.2 Concrete Mixing and Mix Proportions

Nine different mix proportions with the same cement content were designed by using two types of solid wastes from palm oil industry as coarse aggregates. In order to take into account the drying shrinkage strain: the volume and type of aggregates, different curing conditions, different water to cement ratios and partial replacement of OPS by OPBC aggregates were carried out. The two sets of concrete mixes were designed and

OPS concrete without OPBC aggregates was considered as the control concrete. In set (1), the OPS aggregate of the control concrete mix was partially replaced by OPBC in percentages of 10, 20, 30, 40 and 50 by volume. Therefore, the difference between all mixes in the set (1) is only the volume of the OPS and the OPBC used in the concrete mixtures.

The workability of the prepared mixes in set (1) was measured using slump test as shown in Table 6.2. As can be seen in Table 6.2, the incorporation of OPBC in OPS concrete increased the slump value. However, to avoid any segregation of lightweight aggregates the slump value was controlled by reducing the SP content in the mixes containing 40 and 50% OPBC. The reason for higher slump value of OPS-OPBC concretes is due to OPBC is round in shape and has lower water absorption than OPS. The aims to select the same water to cement ratio in set (1) concrete mixes was to observe the effects of OPBC aggregates on workability and the drying shrinkage of the prepared lightweight concretes. Therefore, while increasing the amount of OPBC aggregates in OPS concrete, the workability of the OPS-OPBC concretes was significantly increased without any segregation. After observing the results of workability, it was decided to manage the workability closer to control OPS concrete. For that a set (2) concrete mixes (C-30(2) to C-50(2)) with reduced water to cement ratios from 0.36 to 0.295 were designed. It was observed that by reducing the water to cement ratio the workability was achieved in acceptable range of structural LWAC. Mehta and Monteiro (2006) reported that a structural LWAC with slump value in the range of 50-75 mm was consider as a lightweight concrete with good workability. These slump values are similar to a 100-125 mm of normal weight concrete.

Consequently, in set (2), three concrete mixtures were produced by incorporating 30-50% OPBC aggregates in OPS concrete at an interval of 10%. In this set (2), to control the workability of OPS-OPBC concretes, the water content was reduced instead of reduction in SP content. Therefore, in this set of mixtures the SP content was placed constant but water to cement ratios were reduced to observe the effect of lower water to cement ratio on workability and drying shrinkage. Compared to set (1) concrete mixes, the reduction in the water to cement ratios for C-30(2), C-40(2) and C-50(2) mixes were 10%, 15% and 20%, respectively. It was observed that by reducing the water to cement ratio the slump value was found almost closer to the control OPS concrete (Table 6.2) and found in the acceptable range for the structural concretes.

The concretes were produced in rotating drum mixer. For each mix proportion, the lightweight aggregates (OPS and OPBC) were weighed in dry condition and then pre-soaked for 24 h to control the workability and the effective water content of the concrete. After that the LWAs were air dried in the lab environment for 2-3 h to obtain saturated surface dry condition. The cement and aggregates were placed in the mixer and mixed for 2 min, after that a mixture of 70% mixing water with SP was added to the mixture and mixing was continued for another 3 min. Then the remaining water was added to the mixture and mixing was continued for another 5 min. After that the slump test was performed. All concrete mix proportions are listed in Table 6.2.

Table 6.2: Mix proportions (kg/m^3), slump value (mm) and dry density (kg/m^3) of concretes

Mix group	Mix code	Cement (kg/m^3)	Water (kg/m^3)	w/c	SP (% cement)	Sand (kg/m^3)	Coarse aggregate (kg/m^3)		Slump (mm)	Dry density (kg/m^3)
							OPS	OPBC		
Set (1)	C-0	480	173	0.36	1	890	360	0	55	1790
	C-10	480	173	0.36	1	890	324	51	58	1808
	C-20	480	173	0.36	1	890	288	102	65	1848
	C-30	480	173	0.36	1	890	252	153	90	1833
	C-40	480	173	0.36	0.90	890	216	205	100	1856
	C-50	480	173	0.36	0.85	890	180	256	90	1904
Set (2)	C-30(2)	480	156	0.325	1	890	252	153	65	1852
	C-40(2)	480	149	0.310	1	890	216	205	60	1885
	C-50(2)	480	142	0.295	1	890	180	256	52	1940

6.2.3 Curing Conditions

To determine the effect of the curing conditions on the drying shrinkage of the concrete mixes, the specimens were cured under two following curing regimes.

- Partial early curing (7W): Curing in water for 6 days after demoulding and then air dried in laboratory environment.
- Uncuring (AC): Specimens were kept in laboratory environment after demoulding.

The water for curing had a temperature of 22 ± 2 ºC. The specimens were placed in the uncontrolled room in concrete laboratory with a temperature of 30 ± 2 ºC and a relative humidity of 74 ± 4 %. For 28-day compressive strength test specimens were immersed in water after demoulding till the age of testing.

6.2.4 Experimental Procedure

For each concrete mix, four specimens with size 100 x 100 x 300 mm^3 were prepared to measure the drying shrinkage. After casting, the specimens were covered with plastic sheets and stored in the laboratory environment. The concrete specimens were demoulded after 24 h of casting. For determination of drying shrinkage of all mixes, two specimens were prepared for each curing condition. After demoulding, the specimens were placed in the uncontrolled room. In addition, two specimens for each mixture which were cured in water and placed in uncontrolled room after 7 days of curing. The drying shrinkage was measured by the demountable mechanical strain gauge (DEMEC) with a resolution of 0.001 mm. The DEMEC points were fixed 200 mm apart with V-tech 502 super glue on the three plain sides of the samples and placed in the uncontrolled room for 1-2 hours, so that the applied glue becomes properly dry. After that the initial readings were recorded by the equipment (Gauge type PLR-60-11) as shown in Figure 6.3.

The 28-day compressive strength of all the concretes was selected as average of three cube specimens with 100 mm^3 dimensions. However, the values of the drying shrinkage was obtained using the average measurements of six values for each type of concrete. The average values for test results were selected from the specimens which gave the reliability of about 95±4%.

Figure 6.3: The image of the measurement of drying shrinkage

6.3 Results and Discussion

6.3.1 Compressive Strength

The 28-day compressive strength of all prepared lightweight concretes is shown in Figure 6.4. It was observed that the control OPS concrete (C-0) showed the 28-day compressive strength of about 36 MPa, which falls in the category of normal strength lightweight concrete. As can be seen in the Figure 6.4, the substitution of 10% OPBC aggregates in OPS concrete the compressive strength slightly increased. However, a significant improvement in the compressive strength of OPS concrete was observed when the substitution level of OPBC aggregates was more than 10%. Reduction on the water used in the OPS concrete mixes containing 30 to 50% OPBC further improved the compressive strength of the concretes. The increment in the compressive strength of these concretes (C-30(2) to C-50(2)) was about 20-30%. All these concretes could be considered as high strength lightweight concrete. The highest 28-day compressive strength of 52 MPa was achieved by C-50(2) mixture at 20% lower w/c ratio compared to the mix C-50. This concrete had an oven dry density of about 1940 kg/m^3 which shows that it is around 17% lighter than conventional concrete. The strength grade of this concrete is similar to the 28-day compressive strength of OPS lightweight concrete made with an oven dry density of 1920 kg/m^3 (Shafigh, Mahmud, et al., 2012).

The main reason for the improvement in the compressive strength of OPS-OPBC mixes is that the OPS aggregates are flaky in shape and have smooth surface texture on both concave and convex faces than OPBC aggregate. Due to that the interfacial bond between the aggregates and the cement was very weak and causes a concrete failure under smaller loads (Aslam, Shafigh, Jumaat, et al., 2016). Therefore, the most of the grades of the OPS concrete were within the normal range of lightweight concrete. It was reported by Wilson and Malhotra (1988) that the compressive strength of structural lightweight concrete is directly relevant to the strength of the LWA and the hardened cement paste, as well as the bonding of the aggregate and cement paste in the interfacial zone. However, as can be seen Figure 6.2, the OPBC has more porosity with some holes on the surface. Therefore, compared to smooth textured surface of OPS aggregate there is better interlock between the OPBC aggregates and the cement paste which caused significant improvement on compressive strength of concrete.

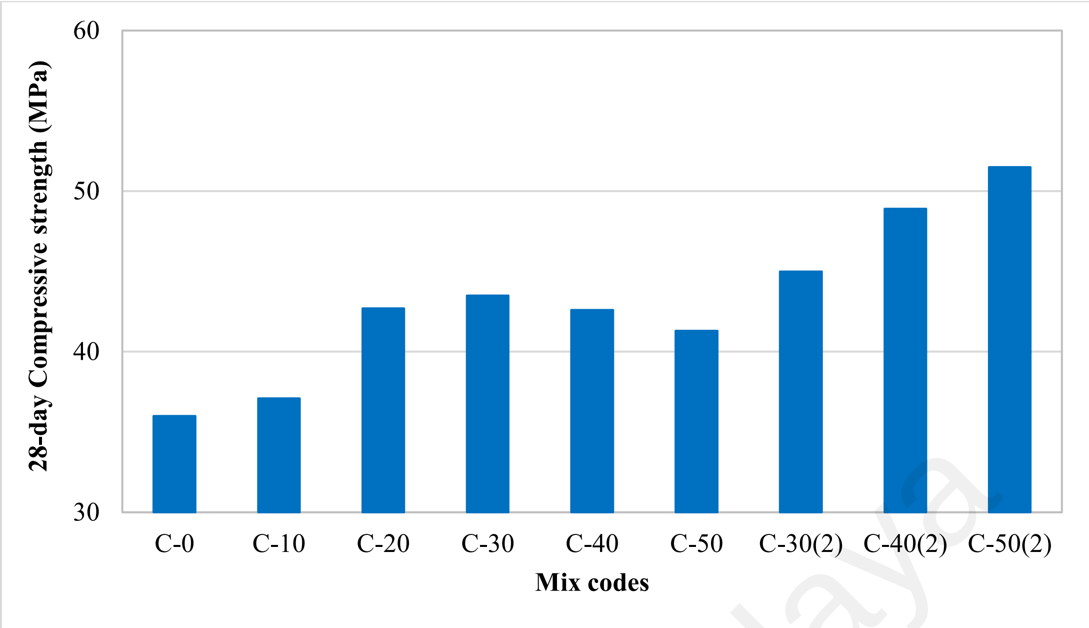


Figure 6.4: 28-day compressive strength of concrete mixes

6.3.2 Specific Strength

Normal weight concrete has low specific strength (compressive strength to density ratio, the 28-day cube compressive strength was used for the computation of the specific strength) compared to steel, which results some economic disadvantages, when being used in multi-story buildings and bridges. However, its efficiency ratio can be improved either by increasing its compressive strength or reducing its density (Aslam et al., 2015). The best alternative to resolve this issue is the use of high strength lightweight concrete. Structural LWAC showed significantly higher specific strength than NWC of the same grade (Mehta & Monteiro, 2006). Moravia, Gumieri, and Vasconcelos (2010) investigated the specific strength and modulus of elasticity of the structural LWAC made of expanded clay aggregate and reported that even though this LWC has a lower compressive strength than NWC, but it has significantly higher specific strength. Figure 6.5 shows the specific strengths of all the concrete mixes. It can be seen that although the contribution of the OPBC aggregates in the OPS concrete increased the dry density of the OPS concrete in the range of 1-8%, however, in most cases, the specific strength of OPS concretes containing OPBC is significantly higher than concrete containing only OPS aggregates (C-0). The results showed that in set (1), compared to control OPS concrete (C-0), the efficiency factor of OPS-OPBC mixes containing 10% OPBC aggregate has no any significant effect. However, the moderate and high volume substitutions significantly increased the efficiency ratios in the range of about 13-15% and 7-12.3%, respectively. While, the specific strength of set (2) concrete mixes was significantly improved in the range of 17-25% by the contribution of 30-50% OPBC aggregates in OPS concrete. The efficiency factor for a NWC with a dry density of 2350 kg/m^3 and 28-day compressive strength similar to mix C-50(2) is 21915 N.M/kg, which is about 17.5% lower than the efficiency of OPS-OPBC concrete. Therefore, it can be said that OPS-OPBC concretes even shows better performance compared to NWC of the same grade. Therefore, it can be concluded that moderate to high volume contribution of OPBC in OPS concrete significantly improve the compressive strength and efficiency of the concrete. The main advantage of using OPBC aggregates in OPS concrete is that it is a waste material and it significantly improved the compressive strength and the efficiency factor of the concrete.

Figure 6.5: Specific strength of concrete mixes

6.3.3 Drying Shrinkage Development

The shrinkage of normal to moderate strength LWC is not usually critical when it is used for fill or insulation purposes, however, in structural use, shrinkage must be considered and is possibly deleterious when it is restrained (Kosmatka et al., 2002). Previous studies (Neville, 2008; Satish & Berntsson, 2003) reported that the drying shrinkage of LWC is greater than NWC and is mostly affected by properties and the quantity of aggregates. In this study, the effect of partially replacement of OPS with OPBC on drying shrinkage of concrete in two sets of concretes was investigated. In the first set of concretes, the pure effect of this substitution was studied. While, in the second set the effect of OPBC contribution as well as reduction of water to cement ratio were investigated together.

6.3.3.1 Drying shrinkage of uncured specimens

The development of drying shrinkage of uncured concrete specimens of mixes illustrated in Figure 6.6 and Figure 6.7. The drying shrinkage of the control OPS lightweight concrete at the age of 234 days was relatively high as 564 micro-strain. Normally, the drying shrinkages of normal concrete are reported to be in the range of 200-800 micro-strain (Zia, Shuaib, & Leming, 1997), while the oil palm kernel shell concrete showed five times the shrinkage value of NWC (Abdullah, 1996). As can be seen in the Figure 6.6, the drying shrinkage of OPS-OPBC concretes is similar to the OPS concrete at early ages. The increasing rate of drying shrinkage of all the mixes was very sharp in the first 28-days. However, the shrinkage value after 56 days was almost constant for all mixes. The mixes C-0 and C-10 showed the same trend of the drying shrinkage up to 3 months, after that the gradually reduction in shrinkage was observed for mix C-10. The mixes of C-20, C-30, C-40 and C-50 showed almost similar trend of the shrinkage from 28-234 days and the shrinkage values observed for these mixes were very close. The mix (C-0) showed similar shrinkage results to all OPS-OPBC mixes in early ages up to 1 month. Although, the similar results were achieved by low (10%) contribution of OPBC up to 3 months. However, at later ages (around 9 months) the OPS concrete on average showed 10% higher shrinkage values compared to OPS-OPBC mixes.

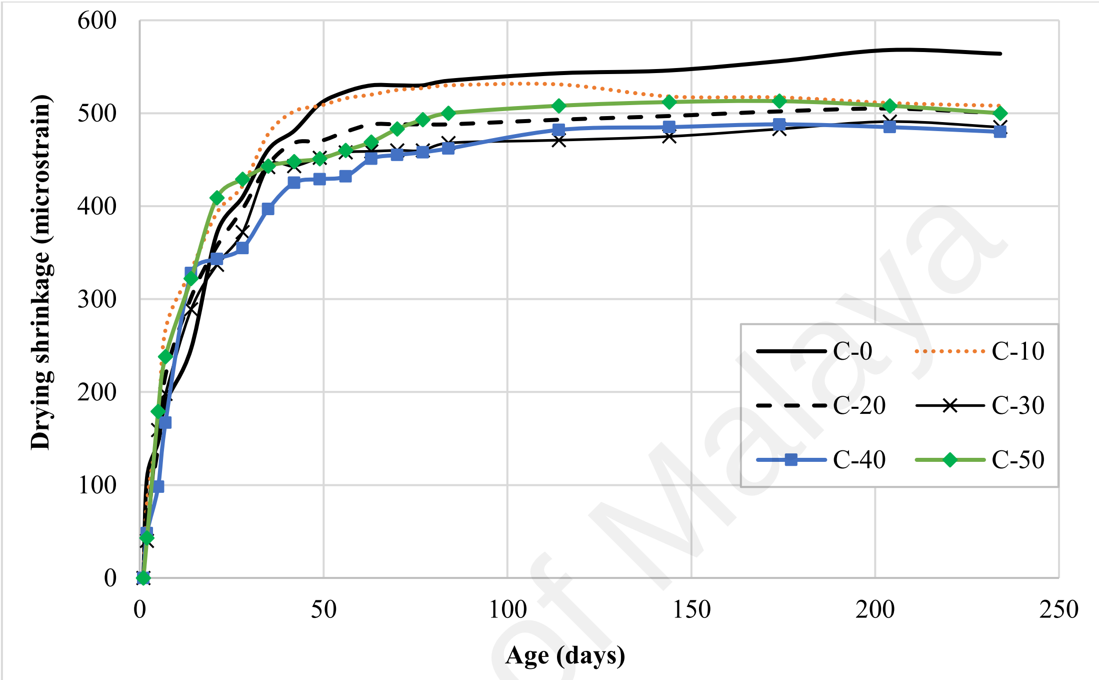


Figure 6.6: Development of drying shrinkage of set (1) concrete mixes

Figure 6.7 shows the results of the drying shrinkage of the set (2) concrete mixes. Compared to set (1) concrete mixes the reduction of w/c ratio for mixes of C-30(2), C-40(2) and C-50(2) was about 10%, 15% and 20%, respectively. As can be seen in Figure 6.7, the drying shrinkage of OPS-OPBC concretes is very similar to the OPS concrete in the first week, but at the later ages a significant reduction in the drying shrinkage was observed in the set (2) concrete mixes. All the three mixes with lower water to cement ratio showed the increasing trend of shrinkage up to 70 days but at later ages the drying shrinkage was almost constant for all mixes. All OPS-OPBC mixes from set (2) showed almost similar results in early ages (28 days) and these values were on average 37% lower than control OPS concrete at the same age. However, at later ages, the mixes C-30(2) and C-40(2) showed lower shrinkage values of about 27% and 35%, respectively compared to the control mix. The mix C-50(2) showed the lowest long-term shrinkage value among all the mixes and it is about 40% lower than the control C-0 mix.

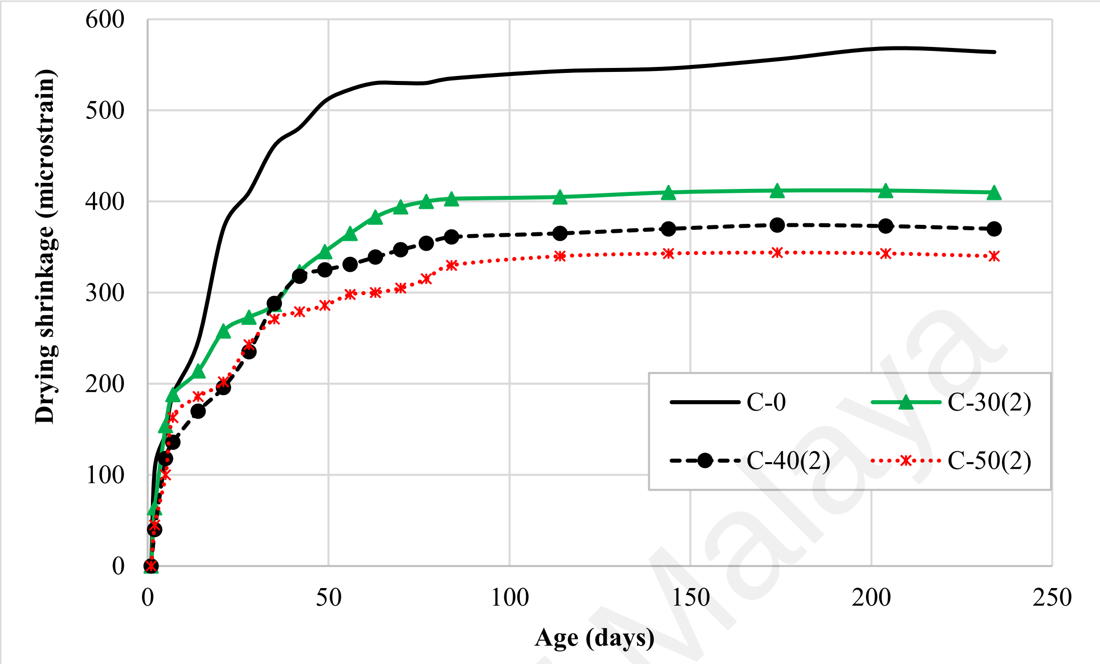


Figure 6.7: Development of dying shrinkage of set (2) concrete mixes

From Figure 6.6, it was observed that the C-0 and C-10 mixes have similar shrinkage values up to age of 3 months. As can be seen in Figure 6.6, OPS concrete mixes containing 20 to 50% OPBC have almost similar drying shrinkage values. A curve of average value of drying shrinkage of these mixes are shown in Figure 6.8. It can be clearly seen from this Figure 6.8, that partial replacement of OPS by OPBC reduced the shrinkage of OPS concrete. This reduction in general was about 13%. The main reason is due to the reduction of cement paste, because the OPBC aggregate have huge porosity on the surface. Therefore, it is expected that the most of the cement paste was absorbed by the higher porosity of the OPBC aggregate. Wongkeo et al. (2012) investigated the compressive strength and drying shrinkage of concretes using multi-blended cement mortars. They reported that generally the shrinkage of concrete initiated from the cement paste.

Similarly, it can be observed from Figure 6.7 that the set (2) concrete mixes (C-30(2) to (C-50(2)) showed very close results to each other at all ages with the average difference of about 14%. Therefore, average values were taken and plotted in Figure 6.8. It can be seen in this Figure 6.8 that the contribution of OPBC in OPS concrete along with reduction on water to cement ratio significantly reduced the drying shrinkage compared to control OPS concrete mix. The average reduction in the drying shrinkage values of the OPS-OPBC concrete mixes (set (2) concrete mixes) was about 34% than the control OPS concrete and about 24% compared to the set (1) concrete mixes containing 20-50% OPBC aggregates. Bogas et al. (2014) reported that the higher the w/c ratio the lower will be the mortar stiffness and higher the volume of evaporable water which cause a significant volume change in the concrete.

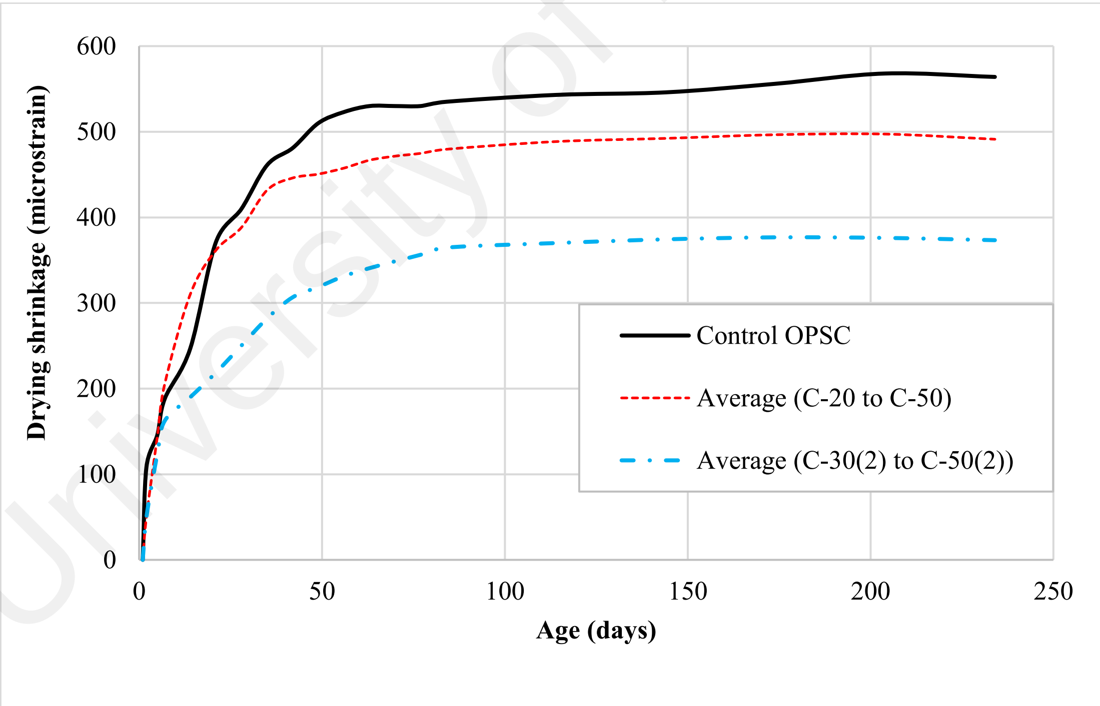


Figure 6.8: Development of dying shrinkage of both sets mixes

There are certain reasons, which possibly play a major role to reduce the drying shrinkage. First reason is the shape and surface texture of the aggregates. It was observed from the experimental results (Figure 6.8) that the OPS concrete is showing the highest drying shrinkage at almost all ages. This might be due to the shape and surface texture of the OPS aggregates; the OPS aggregates are normally flaky in shape with smooth surface texture as compared to the crushed OPBC aggregates. Al-Attar (2008) investigated the drying shrinkage of the ordinary concrete by using crushed and uncrushed gravels as aggregates. He reported that due to smooth texture surface of uncrushed gravels concrete containing these aggregates have more drying compared to crushed aggregate concrete.

The second reason is the type and the stiffness of the aggregates. Previous studies showed that compared to other types of lightweight concrete, modulus of elasticity of OPS concrete is low. It is in the range of 5.31-10.90 GPa (Alengaram et al., 2013; Mahmud et al., 2009). However concrete containing only OPBC as coarse aggregate showed higher modulus of elasticity in the range of 12-26 GPa (Mohammed et al., 2013; 2014). Therefore, it was expected that by using these both aggregates together in one mixture, the modulus of elasticity is improved. It can be verified from the compressive strength results that the incorporation of the OPBC aggregates in OPS concrete significantly increased the strength. The elastic modulus of concrete depends on the moduli of elasticity of its components and their proportions by volume in the concrete (Neville, 1971). Therefore, the main difference between all the prepared mixes is the type and the volume of the coarse aggregates so it can be concluded that the elastic modulus of OPBC grain is higher than the modulus of OPS grain. This is another reason that concrete/aggregate with higher elastic modulus have lower drying shrinkage (Neville, 2008). Shafigh et al. (2012) reported that an expanded clay LWC with a compressive strength of 30% less than OPS concrete showed about 40% higher modulus of elasticity compared to OPS concrete. In this study, the OPS concrete showed higher drying shrinkage results compared to OPS-OPBC mixes. This was majorly due to the improvement in the properties of OPS concrete by the incorporation of OPBC aggregates.

The third reason is the effect of moisture content present in the aggregates. Literature revealed that, several strategies exists to reduce the drying shrinkage, such as use of expansive cements (Saito, Kawamura, & Arakawa, 1991; Weiss & Shah, 2002), addition of shrinkage reducing admixtures (Henkensiefken, Nantung, & Weiss, 2008) and addition of saturate lightweight aggregates (Hemmings et al., 2009; P. Lura et al., 2006). In this study both types of lightweight coarse aggregates were pre-saturated before using in concrete mixture. As can be seen in Table 6.1, OPS have 34% higher water absorption than OPBC aggregates. Therefore, while designing the OPS concrete mixture at the similar water to cement ratio to the OPBC mixtures, the overall water content in the mixture was increased which might be able to create a problem during hydration process. The shrinkage results of the OPS concrete were significantly reduced by the replacement of OPS with OPBC, which might be due to maintained water content in the concrete mixes. Generally, in any type of concrete, if the water content in the mixture will be higher, the mechanical properties will be reduced. Al-Attar (2008) reported that by using saturated coarse aggregates in the concrete mix, it always yields higher drying shrinkage than in dry aggregate mixture. In this study, mostly the mixes which contained higher amount of OPBC aggregates (C-50 and C-50(2)) showed the lowest drying shrinkage. This was due to the moderate internal moisture present in the aggregates which properly performed in the internal curing due to that the properties such as compressive strength and drying shrinkage of the concretes were increased (Figures 6.6 to 6.11).

Another reason might be porosity and the internal curing. Literature revealed (Mannan et al., 2002; Okpala, 1990) that OPS aggregate have smooth nature on both the concave and convex faces, due to that the bond between the shell and cement paste was very weak. However, the crushed OPBC aggregates have higher porosity which causes better interfacial bond with the cement paste. Therefore, it was expected that when the high porous OPBC aggregates were incorporated in OPS concrete it performs better role in the internal curing. During the process of internal curing, saturated LWAs develop a suction pressure in the hydrated cement paste due to the self-desiccation and chemical shrinkage. Henkensiefken et al. (2009) stated that during the process of internal curing, the biggest pores loses water quickly and reduces the capillary stress which increases the hydration process. Therefore, it can be concluded that OPBC with the higher porosity loses internal water quickly, which increases the hydration process of the mixture, due to that the mechanical and durability properties were significantly improved. Bogas et al. (2014) reported that when the internal curing becomes less relevant there is a sudden increase in the shrinkage rate of LWC compared to NWC. They also reported that the shrinkage of LWC depends on the water lost by evaporation. Generally, the LWAs have lower restriction effect on the paste deformations which increases the rate of drying shrinkage.

Figure 6.9 shows the relationship of the drying shrinkage and amount of OPBC aggregates with different proportion levels. It can be clearly seen that as the replacement level increase the drying shrinkage was reduced. This was majorly due to the replacement of OPS by OPBC aggregates in OPS concrete and this higher replacement is protecting majority of the cement paste from drying. It was observed from the experimental results that the OPS concrete showed higher drying shrinkage. As the OPS aggregates were partially replaced by OPBC aggregates at 10% interval, the drying shrinkage was reduced. The mix with 50% replacement of OPS with OPBC gives the lowest drying shrinkage value compared to all the set (1) concrete mixes. This value was about 11.4% lower than control OPS concrete at the long-term ages. However, uncured condition, the set (2) concrete mixes shows sharp reduction rate in shrinkage strain compared to set (1) concrete mixes, due to reduction in water to cement ratio. Kovler and Jensen (2007) reported that if the volume of aggregate replacements increased, the fraction of paste also increases within a specified closeness of a saturated lightweight aggregate. However, when lower replacements were used the fraction of paste was rapidly increase within 1 mm of a saturated LWA particles. Therefore, at low replacement levels, if water in the saturated LWA can travel up to 1 mm than a majority of the paste will be protected (Kovler & Jensen, 2007). While, in higher replacement levels, a larger paste fraction is within 0.2 mm, which might be important at later ages. In this study, it was observed that partial substitution of OPBC aggregates in OPS concrete has a significant effect on the drying shrinkage. Both set mixes showed similar drying shrinkage results to the control concrete up to 7 days, however, the OPS-OPBC mixes of set (1) on average continued the closer trend to the control OPS concrete up to 28 days, after that the shrinkage was reduced. The long-term shrinkage of the OPS concrete was significantly reduced by the contribution of OPBC aggregate. The set (1) concrete mixes showed the reliability of about 55% between the shrinkage results at different OPBC substitutions. However, this ratio for set (2) concrete mixes was very high about 99%. This might be due to reduction in water contents of the aggregates because OPS aggregate have about 39% higher water absorption compared to OPBC. Therefore, the substitution of OPBC in OPS concrete maintains the total amount of water in the mixture. Gribniak, Kaklauskas, Kliukas, and Jakubovskis (2013) reported that the aggregates with high water absorption tend to increase the shrinkage deformations.

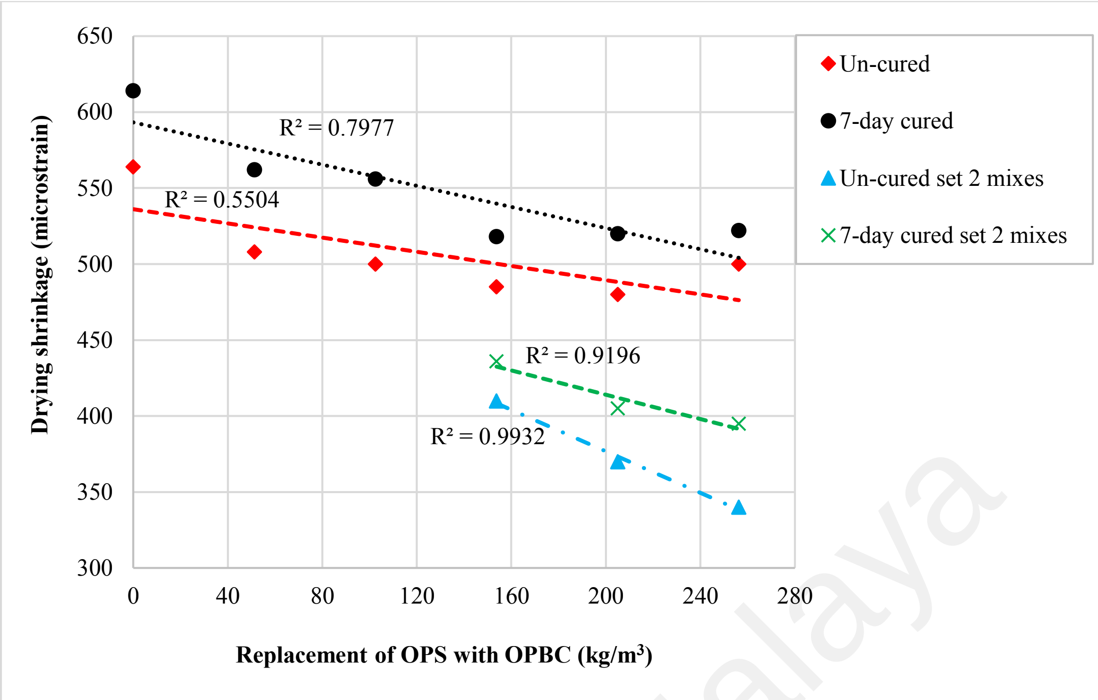


Figure 6.9: 234-days age shrinkage versus the replacement of OPS by OPBC aggregates

In this study, the drying shrinkage was investigated for two sets of concrete mixtures. As discussed earlier, in first set all prepared mixes (C-0 to C-50) contain same water content and were investigated to observe the effect of OPBC aggregate substitutions in the OPS concrete. Although, the small reduction in the drying shrinkage was found in OPS-OPBC mixes compared to the control OPS mixture. This reduction in the shrinkage strain was majorly due to the replacement of the aggregates. After that it was decided to manage the workability closer to control OPS concrete. For that a set (2) concrete mixes (C-30(2) to C-50(2)) with reduced water to cement ratios from 0.36 to 0.295 were designed and drying shrinkage behaviour was investigated. Figure 6.10 shows the relationship between drying shrinkage and water to cement ratios and OPBC content. It can be clearly seen that by reducing the w/c ratio, the drying shrinkage was significantly reduced. The mix C-50(2) at w/c ratio of 0.295 gives lowest drying shrinkage of about 40% under uncured condition compared to OPS concrete at the age of about 9 months. The mixes C-30(2), C-40(2) and C-50(2) showed about 15.5%, 23% and 32% lower drying shrinkage results compared to mixes C-30, C-40 and C-50, respectively from set (1). All these mixes with reduced water to cement ratios on average showed 33.8% lower long-term shrinkage results compared to control OPS concrete. Bogas et al. (2014) reported that for the same cement content, the drying shrinkage increases with increasing w/c ratio and it reduces the mortar stiffness. Therefore, a small reduction of the w/c ratio causes a significant reduction in the drying shrinkage. Beygi, Kazemi, Nikbin, and Amiri (2013) investigated the effect of w/c ratio on the brittleness and fracture parameters of self-compacting concrete. They reported that w/c ratio is the most substantial factors which highly influence the magnitude of the shrinkage of concrete. It was found in this study that a small reduction on the w/c ratio of OPS-OPBC concretes, under air drying curing condition, caused significant reduction in the drying shrinkage.

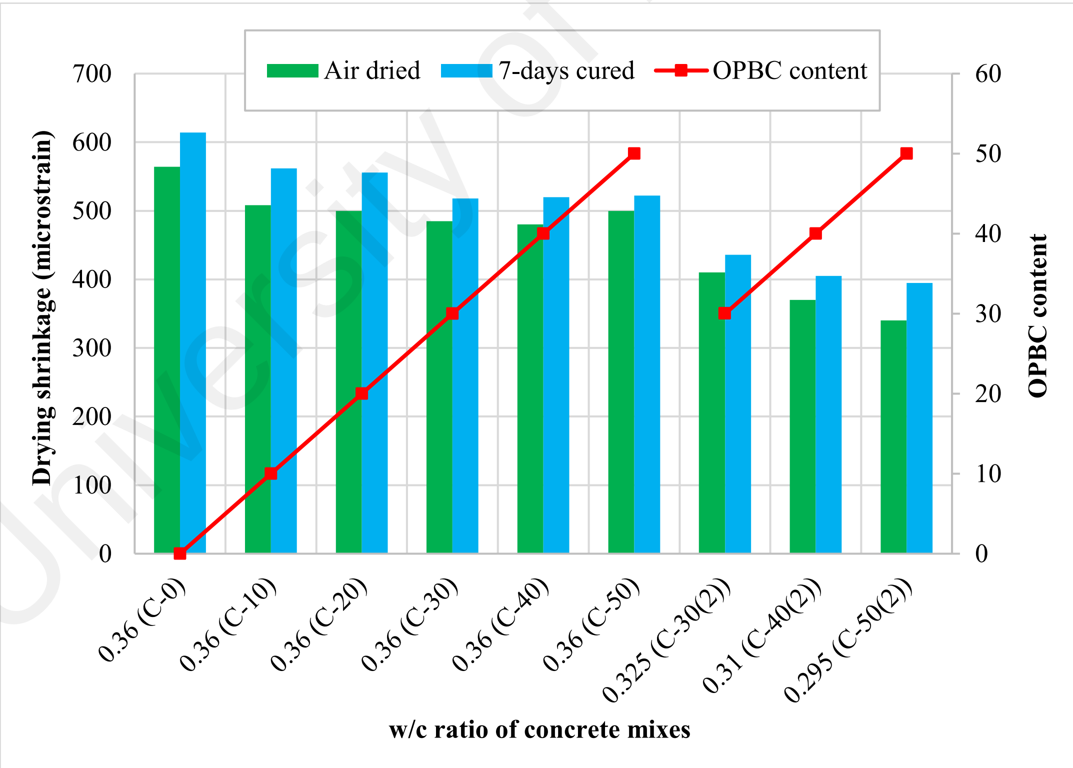


Figure 6.10: Drying shrinkage and OPBC content versus the w/c ratio

6.3.3.2 Drying shrinkage of 7-day cured specimens

The sufficient duration of curing plays a major role to control the shrinkage and initial cracks in concretes Carlson (1938). The longer wet curing rises the efficiency of the materials especially in the early age concrete and also causes a delay and the reduction of the long-term shrinkage (Bogas et al., 2014; Fujiwara, Tomita, & Shimoyamata, 1994). Figure 6.11 and Figure 6.12 illustrated the results of the drying shrinkage of the set (1) and set (2) lightweight concrete mixes under 7-days moist curing up to age of 234 days. Figure 6.11 shows that the control mix (C-0) has the highest drying shrinkage of about 618 mm/mm (micro-strain). In low OPBC substitution levels of 10% and 20%, the shrinkage values are similar at all ages. However, the shrinkage of these two mixes on average are 10% lower than the control OPS concrete. At the medium to high substitution levels (30-50%) the concretes showed almost similar drying shrinkage at all ages. Compared to control mix, these mixes (C-30 to C-50) showed similar shrinkage results up to 7 days, later the shrinkage value was slowly increased up to 28 days, after that a constant range of shrinkage values was achieved. These mixes on average showed 15.3% lower long-term drying shrinkage compared to control OPS concrete.

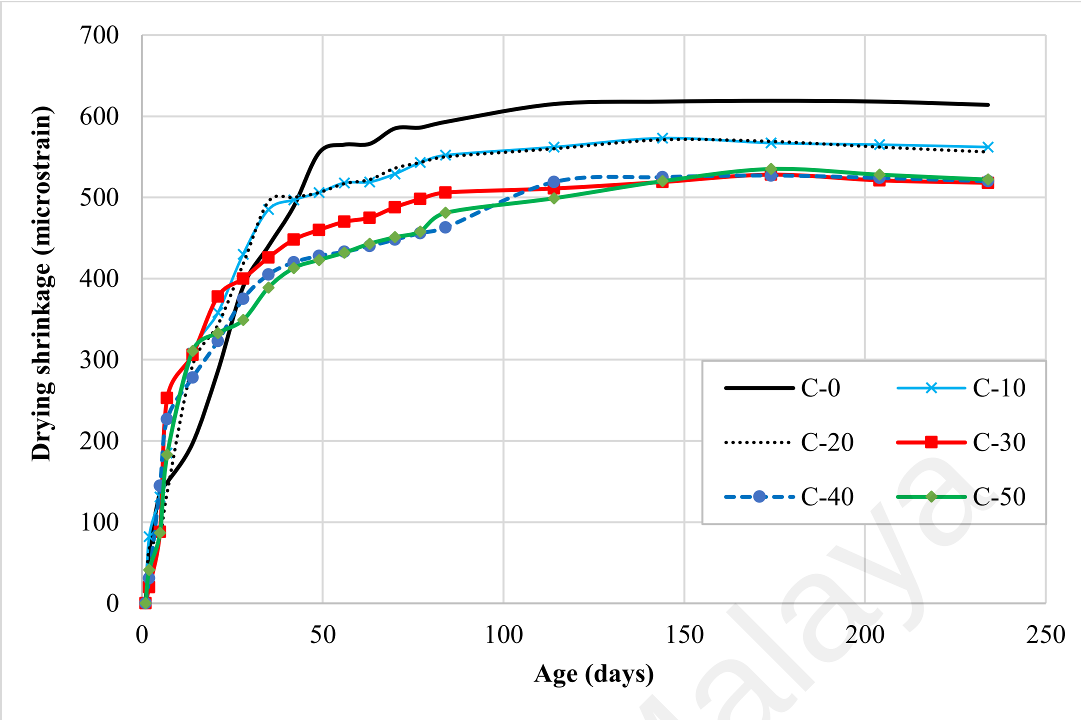


Figure 6.11: Drying shrinkage of OPS-OPBC mixtures after 7 days curing

Figure 6.12 shows the significant effect of reducing of water content on the drying shrinkage of the concretes. The increasing on drying shrinkage for control mix was very sharp up to 50 days. However, it was sharp up to 7-day and gradually increased around 50 days. This shows that if just OPS is used as coarse aggregate in concrete it has great potential for cracking. The shrinkage value of control and OPS-OPBC concrete mixes was almost constant after 80 days. In this age concretes containing OPBC had about 35% less drying shrinkage compared to control mix. Al-Khaiyat and Haque (1998) investigated the LWC using lytag as coarse aggregate with 28-day compressive strength of 50 MPa and fresh density of about 1800 kg/m^3. They inspected the long-term drying shrinkage performance of LWC, when the concrete specimens were cured for 7 days. The drying shrinkage of about 640 micro-strain was reported at the age of 3 months. They demonstrated that the shrinkage behaviour depends on the type of lightweight aggregates. However, Nilsen and Aitcin (1992) investigated the structural LWC using expanded shale aggregate with 28-day compressive strength of 91 MPa. The shrinkage of about 156 micro-strain was achieved at the age of 128 days after 7 days initial moist curing. Similarly, Kayali, Haque, and Zhu (1999) have reported a shrinkage value of about 450 micro-strain at the age of 91 days for structural LWC using sintered fly ash (lytag) as aggregate. They further reported that the shrinkage of NWC was stabilized after 400 days, while shrinkage of LWC did not appear to stabilize after a same period of drying. Compared to the previous studies, in this study, it was observed that without any contribution of OPBC aggregate the control OPS concrete showed drying shrinkage of 595 micro-strain at 90 days age. Although, the low contribution of OPBC further reduced the shrinkage up to 550 micro-strain. However, a significant reduction in the shrinkage was observed by moderate to high contribution of OPBC (set (1) and set (2)) in OPS concrete. This reduction for set (1) and set (2) was about 14.2% and 34.5%, respectively compared to control OPS concrete. Therefore, it can be concluded that the OPS-OPBC mixes shows better results under 7 days curing compared to structural lightweight concretes prepared by lytag, expanded shale and sintered fly ash.

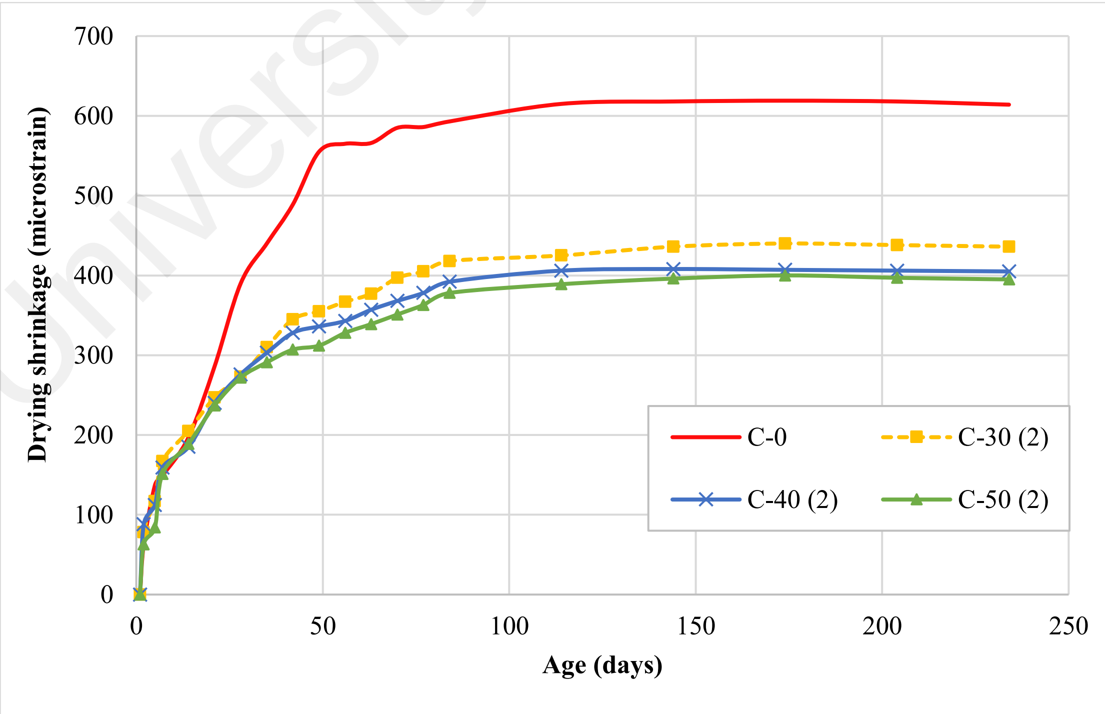


Figure 6.12: Drying shrinkage of OPS-OPBC mixtures at reduced water/cement ratios

Figure 6.13 shows average values of drying shrinkage development of C-30, C-40 and C-50 mixes from set (1) and set (2) concrete mixes. This Figure 6.13 clearly shows the effect of moderate to high volume contributions of OPBC in OPS concrete to reduce drying shrinkage of the concrete. The contribution of 30-50% OPBC in OPS concrete reduced the drying shrinkage on average 15.3%. While at these substitution levels and by reducing the water content the reduction on drying shrinkage of OPS concrete was on average 32.8%.

The set (2) concrete mixes have similar drying shrinkage compared to C-30, C-40, C-50 mixes at the age of 7 days. However, these mixes showed on average 16.5%, 22.4% and 24.3% lower drying shrinkage at later ages. Mix C-50(2) showed the lowest drying shrinkage value. The shrinkage value of this mix was about 40% and 36% lower than OPS control mix for uncured and 7 days cured specimens, respectively.

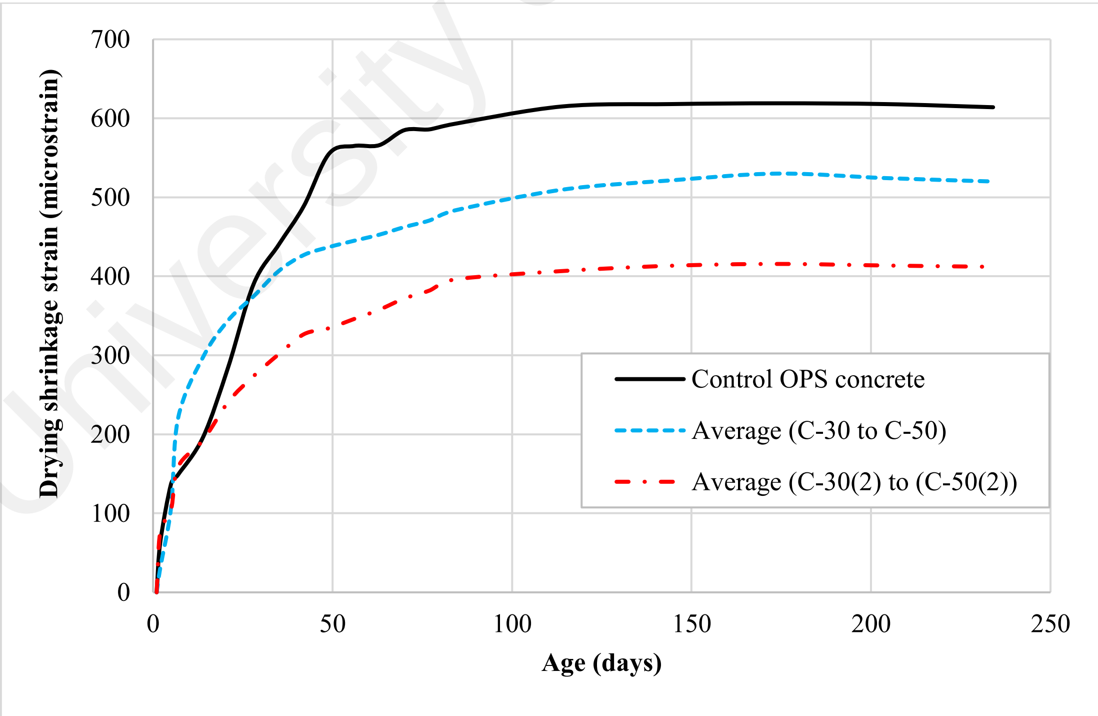


Figure 6.13: Development of dying shrinkage of both set mixes under 7-day curing

Figure 6.9 shows the relationship between the drying shrinkage and amount of OPBC aggregates. As can be seen in this Figure 6.9, the set (1) concrete mixes showed a similar decreasing trend under 7 days curing by the contribution of OPBC aggregates. The mixes with low to moderate contribution showed a sharp decrease in shrinkage results. While, the higher contribution (40-50%) has no any significant effect on the shrinkage. This might be due to similar water contents and 7-days curing. However, the set (2) concrete mixes has shown a significant reduction in the drying shrinkage by the contribution of OPBC aggregates. The mix C-50(2) showed the 14% lower drying shrinkage under air-drying compared to 7-days water curing condition. Therefore, it was observed that the replacement of the aggregates played a major role to reduce the drying shrinkage. Shafigh et al. (2014) investigated the effect of using OPBC sand in OPS concrete on the drying shrinkage of the concrete. They reported that the substitution of the normal sand with OPBC sand had no any significant effect on the drying shrinkage of the OPS concrete. Bogas et al. (2014) studied the NWC prepared by total and partial replacement of normal aggregates with expanded clay lightweight aggregates. They reported that the higher percentage replacement of NWA with expanded clay aggregate increased the long-term shrinkage but the early age shrinkage was reduced due to delay in drying. Further, they stated that the greater the percentage replacement of normal aggregate with LWA the higher the water available to the paste at early ages and it also reduces the restriction effect on the paste deformation. In this study, it was observed that partial substitution of OPBC aggregates in OPS concrete has a significant effect on the drying shrinkage. Both set mixes showed similar drying shrinkage results to the control concrete up to 7 days, however, the long-term shrinkage was significantly reduced by the contribution of OPBC aggregate.

The relationship of drying shrinkage, water to cement ratios and OPBC content is shown in Figure 6.10. It was observed that all the concrete (both sets) mixes containing same volume of the aggregates. However, the major difference between both sets was the w/c ratio. The set (2) concrete mixes with the reduced w/c ratios shows a significant reduction in drying shrinkage compared to mixes with same w/c ratio, this was majorly due to water content and cementitious materials (cement paste). Carlson (1938) investigated the solid specimens of cement paste and concrete made with rubber particles. These particles were porous and non-absorbent. He found that each shrank equally, and concluded that cement paste, if unrestrained by aggregate, would shrink 5 to 15 times more than ordinary concrete. He demonstrated that the water content is the most important single factor which has a significant effect on the shrinkage of concrete. Previous studies (Bogas et al., 2014; Pickett, 1956) revealed that at a constant cement and aggregate contents, the shrinkage is proportional to the w/c ratio, because of the hydrostatic tension in the concrete structure. Picket (1956) established that the original spacing between gel particles depends on the w/c ratio. This was majorly due to the hydrostatic tension (large spaces between particles will become larger and small spaces will become smaller) in the concrete gel structure. He reported that the concretes with higher w/c ratios have a more open structure with large capillaries and have high capacity to shrink as compared to low w/c ratio concretes.

6.3.3.3 Effect of curing condition

Literature revealed that curing has a significant influence on the shrinkage and cracking, therefore a proper curing is required to control the shrinkage reducing effect (Maslehuddin, Ibrahim, Shameem, Ali, & Al-Mehthel, 2013; Tongaroonsri & Tangtermsirikul, 2009). The concrete that is not cured will show higher early-age shrinkage and be prone to cracking. Therefore, it is important to keep moisture on concrete surface for longer than few hours to slow down the drying process of the concrete. Table 6.3 shows the early-age drying shrinkage of all mixes. It can be clearly seen that all the mixes from set (1) shows lower early-age drying shrinkage under 7-days curing. The reduction in the control OPS concrete was higher than OPS-OPBC mixes. The results also confirm that the curing time is essential to minimize the shrinkage at early ages. It causes a lower drying mass loss causing a delaying effect in the development of shrinkage allowing the natural strength development (Oliveira, Ribeiro, & Branco, 2015). While from set (2) concrete mixes, the mixes containing 40-50% OPBC aggregate showed higher early age shrinkage of about 13.5% on average. This increment in early-age shrinkage strain was majorly due to lower water to cement ratio because it causes a proper mixing without any bleeding on the surface of the concrete. West (2010) investigated the process of self-desiccation and autogenous shrinkage. They reported that when no additional water is available during curing of the concrete, moisture is lost because it is consumed by hydration thus shrinkage occurs and majorly it occurs in low water to cement ratio concretes. CCAA (2005) recommended that inadequate bleed water on the surface of concrete will increase the early-age shrinkage and the tendency of the concrete to crack. The major advantage of lower early-age drying shrinkage is very low shrinkage cracks on the concrete surface.

The long-term drying shrinkage of all the mixes are shown in Table 6.4. It is found that the cured specimens have higher shrinkage than the uncured ones, and the difference on average is about 5 % for OPS-OPBC concretes with same water to cement ratio. While this ratio for the reduced w/c ratio concrete mixes was about 9%. Neville (1996) reported that the prolonged moist curing delays the initiation of shrinkage but ultimately has little effect on the magnitude of shrinkage. Carlson (1938) stated the contradictory effects of prolonged moist curing as; it hardens the cement paste and improve the restraining effect against shrinkage; secondly it produces more hydrated cement which causes more shrinkage. It was reported by (Neville, 2008; Mehta & Monteiro, 2006) reported that when the curing period is completed and concrete is subjected to low relative humidity, the resulting gradient acts as a driving force for moisture migration out of the material and causes volume reduction. Similarly, swelling occurs when there is an increase in moisture content due to absorption of water. Therefore, due to the curing of the concrete it resulted higher long-term drying shrinkage. Bogas et al. (2014) studied the comparative behaviour of lightweight and normal weight concretes under 7-days curing. They reported that curing causes a delay and a reduction in both lightweight (16.5%) and normal weight (12.1%) concretes in the long-term shrinkage. Further they demonstrated that in 7 day curing condition, the period in which the shrinkage of LWC is lower than that of NWC was extended from about 3 to 5 months. Moreover, they stated that under real conditions LWC can be quite slow to dry and only the long-term shrinkage should be more meaningful.

Table 6.3: Effect of curing at early ages on the drying shrinkage

Groups	Mixtures	Drying shrinkage (x10^{-6})						Compared average
		Uncured			7 days cured			
		Age (days)			Age (days)			
		3	7	14	3	7	14	%
Set (1)	C-0	108	188	247	63(-42%)	149(-21%)	197(-20%)	-28
	C-10	82	267	331	82(+0%)	185(-31%)	308(-7%)	-13
	C-20	68	218	302	67(-1%)	137(-37%)	293(-3%)	-14
	C-30	40	198	289	20(-50%)	253(+28%)	306(+6%)	-5
	C-40	48	167	328	31(-35%)	227(+36%)	278(-15%)	-5
	C-50	43	238	322	41(-5%)	183(-23%)	311(-3%)	-10
Set (2)	C-30(2)	64	188	214	78(+22%)	167(-11%)	205(-4%)	+2
	C-40(2)	40	136	170	88(+20%)	159(+17%)	185(+9%)	+15
	C-50(2)	45	163	186	63(+40%)	151(-7%)	189(+2%)	+12

(-) decrease in shrinkage due to 7 days curing, (+) increase in shrinkage due to 7 days curing

Table 6.4: Effect of long-term cure on the drying shrinkage

Groups	Mixtures	Drying shrinkage (x10^{-6})								Compared average
		Uncured)				7 days cured				
		Age (days)				Age (days)				
		28	84	174	234	28	84	174	234	%
Set (1)	C-0	410	535	556	564	390(-5%)	593(+11%)	619(+11%)	614(+9%)	+7
	C-10	422	530	517	508	430(+2%)	552(+4%)	567(+10%)	562(+11%)	+7
	C-20	397	488	502	500	418(+5%)	550(+13%)	569(+13%)	556(+11%)	+11
	C-30	372	468	483	485	400(+8%)	506(+8%)	528(+9%)	518(+7%)	+8
	C-40	355	462	488	480	375(+6%)	463(+1%)	527(+8%)	520(+8%)	+6
	C-50	429	500	513	500	349(-19%)	481(-4%)	535(+4%)	522(+4%)	-4
Set (2)	C-30(2)	273	403	412	410	273(+0%)	418(+4%)	440(+7%)	436(+6%)	+4
	C-40(2)	235	361	374	370	276(+17%)	392(+9%)	407(+9%)	405(+9%)	+11
	C-50(2)	243	330	344	340	272(+12%)	378(+15%)	400(+6%)	95(+16%)	+12

(-) decrease in shrinkage due to 7 days curing, (+) increase in shrinkage due to 7 days curing

6.4 Conclusions

Oil palm shell (or palm kernel shell) lightweight aggregate concrete has relatively high short and long terms drying shrinkage and it can only be reduced by the reduction of OPS volume in the concrete. For this purpose, in this study, OPS was partially replaced with another solid waste from palm oil industry, namely oil-palm-boiler clinker (OPBC). Based on the test results, the following conclusions can be drawn:

1. The increase in the amount of OPBC aggregates in OPS concrete increased the slump value and dry density of the OPS concrete.
2. The contribution of OPBC aggregates in OPS concrete increased the specific strength of the concrete.
3. In continuous moist curing, the set (1) concrete mixes containing OPBC from 20 to 50%, the compressive strength of grade 30 OPS concrete could be transformed to grade 40. However, the set (2) concrete mixes with 30-50% OPBC aggregate had significant improvement in the compressive strength and grade 50 OPS lightweight concrete was achieved.
4. In set (1), the OPS-OPBC mixes showed similar shrinkage results to the control OPS concrete at early ages. However, the long-term shrinkage was significantly reduced compared to control OPS concrete.
5. The contribution of OPBC in OPS concrete along with reduction on water to cement ratio significantly reduced the drying shrinkage compared to control OPS concrete. The mix C-50(2) showed the lowest shrinkage value among all the mixes and it was about 40% lower than the C-0 mixture.
6. The OPS-OPBC concrete mixes shows better results under 7-day moist curing compared to structural lightweight aggregate concretes were made of lightweight aggregates such as lytag, expanded shale and sintered fly ash.

7. All the set (1) concrete mixes showed lower early-age drying shrinkage under 7-day curing compared to uncured specimens. The results also confirm that the curing time is essential to minimize the shrinkage at early ages. Therefore, the pre-saturation of aggregates reduces the early-age shrinkage and delays the drying shrinkage. While in set (2) concrete mixes, the mixes containing 40-50% OPBC aggregate showed higher early age shrinkage, on average, about 13.5% under 7-day curing compared to uncured specimens.

CHAPTER 7: EFFECT OF COARSE LIGHTWEIGHT AGGREGATE TYPE ON THE DRYING SHRINKAGE DEVELOPMENT OF CONCRETE

7.1 Introduction

The knowledge of the shrinkage characteristics in the concrete is a necessary beginning point in the design of structures for crack control. It helps the designer to evaluate the probable shrinkage development in prestressed and reinforced concrete members and the suitable steps can be taken in design to accommodate this movement (CCAA, 2002). Similar to the hydration of cement, shrinkage is also an everlasting process when the concrete is exposed to drying. It tends to reach an equilibrium with that environment which causes evaporation of moisture from the gel pores and causes the change in volume (CCAA, 2002). The rate of evaporation depends on several factors and ingredients such as temperature, relative humidity, cement paste content, water to binder ratio and the area of exposed surface of concrete. The higher the paste volume is, the higher will be the drying shrinkage of concrete (Bissonnette, Pierre, & Pigeon, 1999). However, the fineness and the type of cement do not have an important effect on shrinkage (Neville, 1996). Several researchers (Hansen, 1987; Neville, 1996) reported that the water to cement ratio determines the porosity of concrete due to that it has an important influence in the shrinkage process. Neville (1996; 2008) stated that the shrinkage of cement paste is directly proportional to the water to binder ratio when it lies between 0.20 to 0.60.

The aggregates are the most important material in relation with shrinkage, it restricts the cement paste contraction. Carlson (1938) was the pioneer to propose the restraining effect of the aggregates on the shrinkage of concrete. However, the rigidity of an aggregates has a significant effect on the shrinkage, the more the aggregate are rigid, the less will be the shrinkage in concrete. Nevertheless, some of the aggregates also shrink on drying, such those from some sedimentary rocks (Neville, 1996). Several researchers

(Carlson, 1938; Pickett, 1956; Powers, 1959; Rao, 2001) reported that an increase in aggregate size and contents increases the volume of aggregate which provides restraint in concrete and causes reduction in shrinkage. Zhang, Zakaria, and Hama (2013) investigated the influence of aggregate materials on the drying shrinkage properties of concrete using five types of fine aggregates such as natural sand, copper slag sand, limestone sand, hard sandstone and blast furnace slag sand. They found the maximum drying shrinkage strain in the range of 650-790 micro-strain at the age of 90 days. Shafigh, Mahmud, et al. (2014) investigated drying shrinkage behaviour of OPS lightweight concrete in that the normal sand was replaced with oil-palm-boiler clinker (OPBC) sand from 0 to 50%. They reported that the incorporation of OPBC sand in OPS concrete does not have a significant effect on the drying shrinkage strain of OPS concrete. Carlson (1938) investigated the effect of coarse aggregate on drying shrinkage of concrete and reported that aggregates with high porosity and water absorption have usually low elastic modulus, which will result in less restraint against shrinkage of concrete. Whereas, Matsushita and Tsuruta (1998) investigated the effects of normal coarse aggregate types such as Crystalline Schist, Amphibolite, and Andesite on the autogenous shrinkage of concrete. They reported that if the coarse aggregate volume was maintained constant, it negligibly affects the autogenous shrinkage of high-strength concrete.

In the case of lightweight aggregate concrete (LWAC), the drying shrinkage is greater than conventional concrete and is mostly affected by the properties and the volume of aggregates (Satish & Berntsson, 2003). Lura, Van Breugel, and Maruyama (2002) reported that the use of lightweight aggregates (LWA) in the concrete mixes reduces the self-desiccation of the cement paste. They described that LWAC showed lower drying shrinkage at the early ages, however, at later ages, the rate of shrinkage strain was higher compared to NWC. It was due to lower modulus of elasticity of LWA and offering less restraint to shrinkage of the paste. Selih and Bremner (1996) stated that the adsorbed water in LWAs was latterly released into the cement paste due to the internal curing, which compensates for the initial water lost by drying and self-desiccation. Due to the continuous hydration of cement paste and the internal curing, the deformation resistance of concrete matrix is higher, and, therefore, less water is available for evaporation. Several researchers (Coquillat, 1986; Shafigh et al., 2014; Hossain & Lachemi, 2007) by considering multi effective factors on drying shrinkage of concrete together with different variability of LWAs, and concluded that LWACs have higher drying shrinkage compared to NWCs. However, some reports showed that shrinkage in LWAC is less than or similar to shrinkage of NWC (Shafigh et al., 2014; Wegen & Bijen, 1985). For general design work, it was suggested that the shrinkage of LWAC is between 1.4 and 2 times that of NWC (Clarke, 2002).

Oil palm shell (OPS) and oil-palm-boiler clinker (OPBC) are reported to be the most growing agricultural solid waste materials from the oil palm industry in tropical regime countries such as Malaysia, Indonesia, Nigeria and Thailand. Recently, because of the growing environmental concerns, the recycling of the waste as a construction material appears to be a feasible solution to resolve the pollution and economical problems to design the green and sustainable buildings (Raut, Ralegaonkar, & Mandavgane, 2011). OPS and OPBC as an alternative aggregate were investigated by the number of researchers to produce structural lightweight aggregate concrete (Aslam et al., 2016; Shafigh et al., 2013; Zhang et al., 2013). However, several researchers (Ayhan, Gonul, Gonul, & Karakus, 2011; Bhatty & Reid, 1989) preferred to used artificial lightweight aggregates such as expanded clay and lytag to produce structural lightweight aggregate concrete. Production of artificial LWAs or synthetic products requires huge combustion fuels and high temperatures (1100-1200 °C). Due to that, these types of lightweight aggregates are not ecological and economical materials (Ayhan et al., 2011; Bhatty &

Reid, 1989; Harmon, 2007). In addition, artificial LWAs (LECA) as an aggregate is not locally available and should be imported to the country, however, it is expensive.

The main aim of this study is to investigate the effect of replacement of conventional aggregate with lightweight solid wastes such as OPS and OPBC and comparison with drying shrinkage of lightweight expanded clay lightweight aggregate concrete.

7.2 Experimental Programme

7.2.1 Materials

All the materials used in this study were collected and produced in the Malaysia, except expanded clay lightweight aggregate. The ordinary Portland cement (OPC) of grade 48 was used as a binder in this investigation, conforming to MS522, part-1:2003. The specific gravity, Blaine surface area, initial and final setting times of the cement were 3.14, 3510 cm^2/g, 65 min and 140 min, respectively.

The aggregates including crushed granite (NWA), oil palm shell (OPS), oil-palm-boiler clinker (OPBC) and lightweight expanded clay (LECA) with the nominal size of 8-12 mm were used as coarse aggregates in the concrete mixtures. During preparation, the OPS aggregates were stored at open atmosphere for about 6-8 months to remove the fibers from the surface (Aslam et al., 2016). After that the aggregates were washed using the detergent powder to remove the oil and impurities from the surface. While, the lumps of the OPBC aggregates were crushed using a crushing machine. After washing and crushing, both aggregates were sieved to get the similar grading to the expanded clay and the conventional aggregates. The properties of those four types of coarse aggregates are shown in Table 7.1, and their shapes are also mentioned in Figure 7.1. The locally available mining sand with the maximum grain size, specific gravity, fineness modulus and 24-h water absorption of 4.75 mm, 2.7, 2.9 and 1.2%, respectively, was used as fine aggregate.

Water which was free from injurious amount of deleterious materials was used for mixing and curing. While, the Sika Viscocrete with the total chloride ion content of ≤ 0.1 and compatible with all type of cements according to BS-5075 was used as super-plasticizer (SP) to control the workability of the concrete.

Table 7.1: Physical properties of the aggregates

Properties	Coarse aggregates			
	NWA	OPS	OPBC	LECA
Specific gravity	2.65	1.19	1.69	0.64
Density (compacted) (kg/m^3)	1490	610	860	475
24-h water absorption (%)	<1	20.5	7.0	28

Figure 7.1: Types of coarse aggregates (a) Oil-palm-boiler clinker (OPBC), (b) Crushed granite, (c) Oil palm shell (OPS) and (d) Lightweight expanded clay aggregate (LECA)

7.2.2 Mix Proportions and Mixing Procedure

Four different concrete mixes were prepared with a main difference of coarse aggregate. Table 7.2 shows mix proportions of concretes. The concrete mixes were prepared in the rotating drum mixer. Initially, all the materials were weighed in dry condition, the OPS, OPBC and LECA aggregates were pre-soaked in water for 24 hours to control the effective water content of the concrete. After that the pre-soaked lightweight aggregates were air dried in the lab environment for 2-3 hours to obtain saturated surface dry condition. For each mix proportion, the cement and aggregates were placed in the mixer and mixed for 2 minutes. Then the mixture of 70% mixing water with SP was added to the mixer and mixing was continued for another 3 minutes. After that the remaining water was added to the mixture and mixing was continued for another 5 minutes. Then, slump test was conducted.

It was observed that substitution of normal coarse aggregate with lightweight aggregates reduced the slump value. The reduction was more significant for OPS and OPBC concretes. Therefore, SP dosage was slightly increased for these mixtures.

Table 7.2: Concrete mix proportions

Mix code	Cement (kg/m^3)	Water (kg/m^3)	SP (% cement)	Sand (kg/m^3)	Coarse aggregate (kg/m^3)				Slump (mm)	Oven dry density (kg/m^3)	28-day compressive strength (MPa)
					OPS	OPBC	LECA	NWA			
NWC	480	173	0.85	890	-	-	-	802	115	2285	70
OPSC	480	173	1	890	360	-	-	-	55	1790	36
OPBC	480	173	1	890	-	460	-	-	85	2050	57
LECA	480	173	0.85	890	-	-	194	-	90	1590	25

7.2.3 Curing Conditions

To determine the effect of the curing conditions on the drying shrinkage of the concrete mixes, the specimens were cured under two different curing regimes of air drying (AC) in that the specimens were kept in laboratory environment after demoulding, and partial early curing (7W) in that the specimens were cured in water for 6 days then dried in air under laboratory environment. The curing water has an average temperature of 22 ± 2 ^oC, while, the uncontrolled room in the laboratory had a temperature of 30 ± 2 ^oC and a relative humidity of 74 ± 4 %.

7.2.4 Specimens Size and Testing Procedure

Four specimens with the size of 100 × 100 × 300 mm were prepared to measure the drying shrinkage strains of all the designed concrete mixes. After moulding, the specimens were covered with plastic sheets and stored in the laboratory environment. The specimens were demoulded after 24 h of the casting. Two specimens were prepared for each mix to obtain the drying shrinkage strain under the two curing conditions. The drying shrinkage strains were measured by demountable mechanical strain gauge (DEMEC) with a resolution of 0.001 mm. The DEMEC points were fixed 200 mm apart with V-tech 502 super glue on the three plain sides of the specimens and placed in the uncontrolled room for about one hour, so that the applied glue becomes properly dried, the initial readings were recorded by the equipment (Gauge type PLR-60-11).

7.3 Results and Discussions

7.3.1 Drying Shrinkage Development

The drying shrinkage of normal to moderate strength LWCs plays very important role in the structural applications, it must be considered and is possibly dangerous when it is restrained (Kosmatka et al., 2002). Most of the studies (Neville, 2008; Satish & Berntsson, 2003; Aslam et al., 2016) revealed that the shrinkage strain of LWC is higher than the NWC and is mostly affected by the quantity and the properties of the aggregates. Therefore, in this study, the effect of different types of coarse aggregates such as OPS, OPBC, LECA and crushed granite on the drying shrinkage of the concretes was investigated.

7.3.1.1 Drying shrinkage of uncured specimens

The drying shrinkage strain development of all the prepared concrete mixes is shown in Figure 7.2 until the age of one year. All the concrete mixes have showed consistent development of the drying shrinkage. Among the prepared concrete mixes, the conventional concrete mixture (mix NWC) was considered as the bench mark to compare its shrinkage development behaviour with the lightweight aggregate concretes. As can be seen in Figure 7.2, after one year of drying, the mix NWC showed the drying shrinkage of about 320 micro-strain. However, at the same age, the concrete mixes of OPSC, OPBC and LECA showed the drying shrinkage values of about 570, 360 and 345 micro-strains, respectively.

Zia et al. (1997) reported that generally the drying shrinkage of the conventional concrete found in the range of 200-800 micro-strain, while, Abdullah (1996) reported that OPS concrete has five times the higher shrinkage strain than the NWC. The OPS concrete (mix OPSC) showed the highest drying shrinkage strain, which is about 44%, 39% and 36% higher than the shrinkages of mixes NWC, LECA and OPBC, respectively. However, compared to mix NWC, the mixes LECA and OPBC showed very close results of the drying shrinkage at almost all the ages. The difference between the shrinkage values of the mix NWC and LECA and OPBC mixes was found in the range of 7-12%.

As can be seen in Figure 7.2, all the lightweight concrete mixes showed the closer results of the drying shrinkage to normal weight concrete at early ages (first week). However, up to 28 days age, the mix OPSC showed much higher increment in the drying shrinkage compared to the other mixes. This is because, the increasing rate of the shrinkage of mix OPSC was very sharp in the first 28-days, whereas, at the same age, the remaining mixes showed a consistent increment in the drying shrinkage. However, the drying shrinkage values of mix OPSC was sharply increased up to 56 days age, then, it was almost constant.

The mixes of NWC, LECA and OPBC showed almost similar trend of the shrinkage from 7 to 365 days and the shrinkage values observed for these mixes were very close. At 28-days age, the average difference between these mixes was found about 10%.

However, at later ages (365 days) the OPS concrete on average showed 40% higher shrinkage values compared to the other mixes.

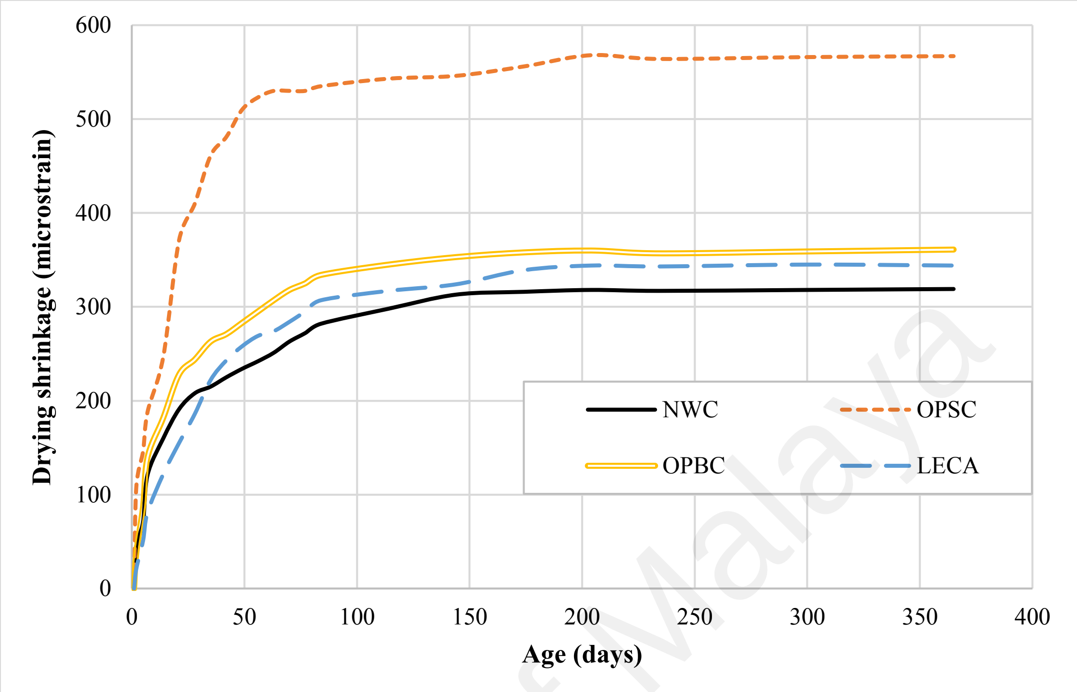


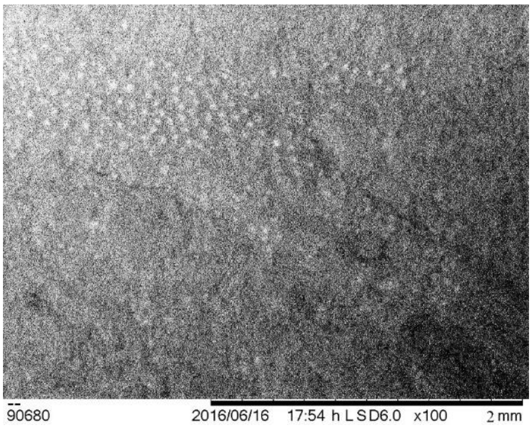
Figure 7.2: Development of drying shrinkage strains under air-drying condition

There are certain reasons, which possibly play a major role in the drying shrinkage strain, among that the type of coarse aggregate plays most critical role in restraining the shrinkage strain of the concrete. First reason is the shape and surface texture of the aggregates. Kong and Evans (1998) reported that the shape and surface texture of the coarse aggregate influence the loss of moisture and it has an indirect effect on the drying shrinkage of the concrete. As can be seen in Figure 7.2 that the mix OPSC showed the highest drying shrinkage strain at almost all ages compared to the other mixes. This might be due to the surface texture and the shape of the OPS aggregates. Figure 7.3 shown the scanning electronic microscopy (SEM) images of all types of aggregates. As can be seen in figure, the OPS aggregates have smooth surface texture compared to the crushed granite, crushed OPBC and uncrushed expanded clay aggregates. Meanwhile, the surface texture of the aggregates can also be seen in Figure 7.1. Therefore, it is expected that the bonding between the OPS aggregate and the cement paste was not that much higher as compared to other concretes. This might be the reason of higher drying shrinkage in the OPS concrete. Al-Attar (2008) studied the shrinkage strain of the conventional concrete using crushed and uncrushed gravels as coarse aggregates. He reported that due to smooth texture surface of uncrushed gravels concrete containing these aggregates have more drying shrinkage compared to crushed aggregate concrete.

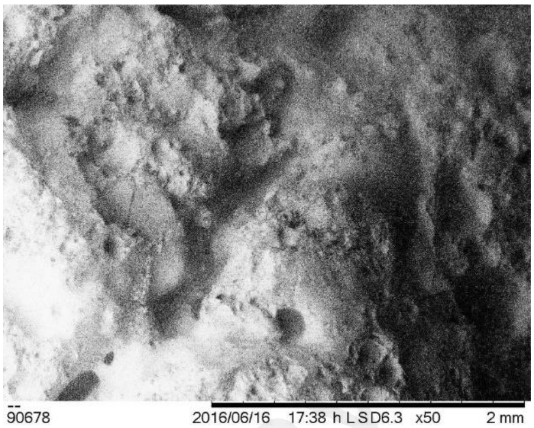
The second reason is the strength and the toughness of the aggregates. Chandra and Berntsson (2002) reported that the shrinkage strain of LWAC is mostly affected by the properties of the LWA and the amount of the aggregate contents. This is because almost 75% of the volume of concrete is made up of aggregate. In addition, Mehta and Monteiro (2006) reported that the modulus of elasticity of the aggregate is a more important factor than the other characteristics of the aggregate. It can control the deformation of concrete. Previous studies showed that the modulus of elasticity of the OPS lightweight concrete is generally very low, found in the range of 5.30-11 GPa (Alengaram et al., 2013; Mahmud et al., 2009). Whereas, the modulus of elasticity for OPBC concrete was found in the range of 12-26 GPa (Mohammed et al., 2013; 2014), structural LWC using artificial lightweight aggregate was found between 10 to 24 GPa (CEB/FIP, 1977), and normal weight concretes in the range of 14-41 GPa (Wilson & Kosmatka, 2011). Neville (2008) reported that the concrete/aggregate with higher elastic modulus have lower drying shrinkage. Higher drying shrinkage of OPS concrete shows that this aggregate has lower modulus of elasticity compared to the other aggregates. Aslam et al. (2016) reported that the modulus of elasticity of OPS concrete was increased in the range of 18-24% by the incorporation of OPBC aggregates. As can be seen in Table 7.1, the normal and OPBC concretes showed about 49% and 37%, respectively higher the 28-day compressive strength compared to the mix OPS. This is due to normal OPBC grains are tougher, which resulted lower drying shrinkage for the OPBC concrete.

LECA concrete showed lower drying shrinkage compared to the OPS concrete, although, it has lowest 28-day compressive strength compared to the other concrete mixes. This was also due to higher modulus of elasticity of a LECA grains compared to an OPS grains. It was reported by (Shafigh, Jumaat, Mahmud, et al., 2012; Shafigh, Mahmud, et al., 2012) that the LECA lightweight concrete with 30% lower 28-day compressive strength than OPS concrete showed about 40% higher modulus of elasticity compared to OPS concrete. Therefore, it can be concluded that the modulus of elasticity of aggregate has more influence to drying shrinkage of concrete compared to compressive strength of the aggregate.

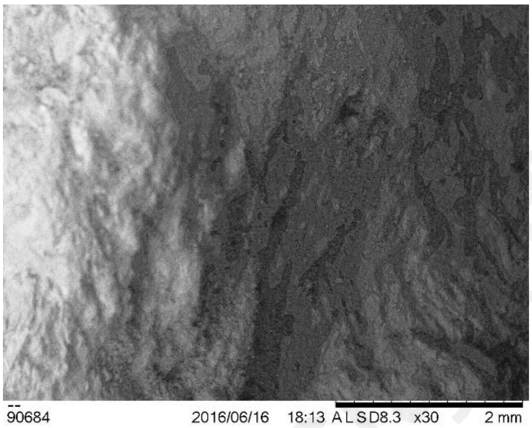
The mix P showed higher drying shrinkage compared to shrinkage strain of the OPBC, LECA and NWC mixes. This was majorly due to the formation of micro-cracks during the hardening of the cement paste in concrete. Figure 7.4 shows the micro-cracks pattern in the OPS lightweight concrete during the hardening of the cement paste. Gao, Lo, and Tam (2002) reported that the main reason for the lower modulus of elasticity in high performance LWACs is the formation of micro-cracks during the hardening of the cement paste in concrete mixture. Due to the higher drying shrinkage of the mix OPS, as well as the weak bonding between OPS and cement paste resulted micro-cracks in the interfacial transaction zone (ITZ), and consequently reduces the mechanical properties such as modulus of elasticity and compressive strength. Further, it can also be clarified that due to the development of such micro-cracks in ITZ, the compressive strength of the OPS concrete is more sensitive to the lack of curing. Previous studies (Aslam et al., 2016; Shafigh et al., 2012; 2013) reported that the OPS concretes are very sensitive to the lack of curing, particularly, when the cementitious materials were used in the concrete mixture. However, compared to mix OPS, the aggregates in other concrete mixes (OPBC, LECA and normal weight concrete) showed better bonding with the cement paste in the ITZ and reduced the formation of micro-cracks, which resulted significantly lower drying shrinkage strains.



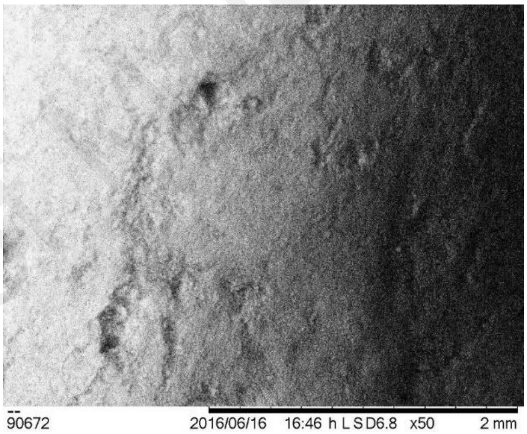
(a) Oil palm shell



(b) Oil-palm-boiler clinker



(c) Lightweight expanded clay aggregate



(d) Crushed granite

Figure 7.3: Scanning Electronic Microscopic image of the aggregates

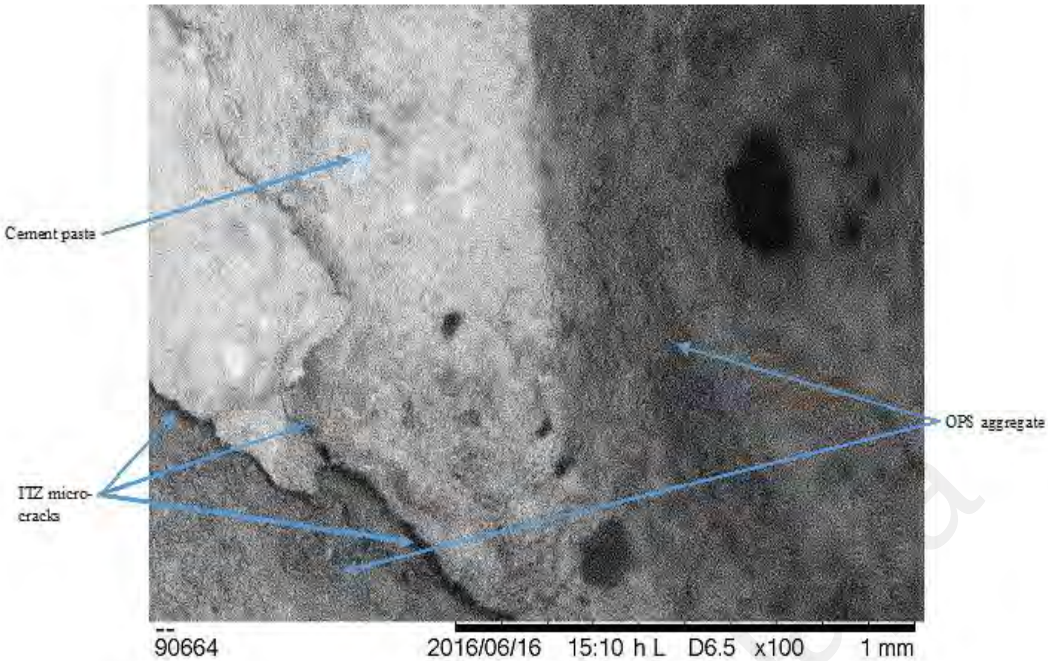


Figure 7.4: A micro-cracks occurred during the drying of OPS concrete

7.3.1.2 Drying shrinkage of cured specimens

In term of shrinkage, the duration of curing plays an important role to identify the initial cracks in the concretes (Carlson, 1938). The longer curing duration increases the efficiency of the materials, especially, in the early ages and also causes a delay in the long-term shrinkage (Bogas et al., 2014). The shrinkage strain development for the prepared concrete mixes after 7-days moist curing is shown in Figure 7.5 until the age of one year. Similar to the uncured specimens, all the concrete mixes have shown consistent development of the drying shrinkage. As can be seen in Figure 7.5, the mix NWC showed the drying shrinkage of about 330 micro-strain at the age of 365 days. However, at the same age, the concrete mixes OPSC, OPBC and LECA showed the drying shrinkage values of about 615, 375 and 365 micro-strains, respectively. It was found that OPS concrete (mix OPSC) showed the highest drying shrinkage strain, which is about 47%, 41% and 39% higher than the shrinkages of mixes NWC, LECA and OPBC, respectively. However, the concretes prepared with oil-palm-boiler clinker (OPBC) and expanded clay aggregates also showed higher drying shrinkage values compared to the conventional concrete. The average difference between the shrinkage values of the mix NWC and mixes of LECA and OPBC was in the range of 10-13% for all ages.

As can be seen in Figure 7.5, all the concrete mixes showed the closer results of the drying shrinkage at early ages (up to 7-days). However, from 7 to 50 days the drying shrinkage of OPS concrete was sharply increased, the rate of increment in shrinkage for this concrete was very high, and from 50 to 365 days age it slowly increased and changed to the constant line. The mixes NWC, LECA and OPBC showed almost similar and the consistent drying shrinkage results up to 75 days age, however, at later ages (365 days), the average difference between drying shrinkage of mixes N, L and C was found about 11.5%.

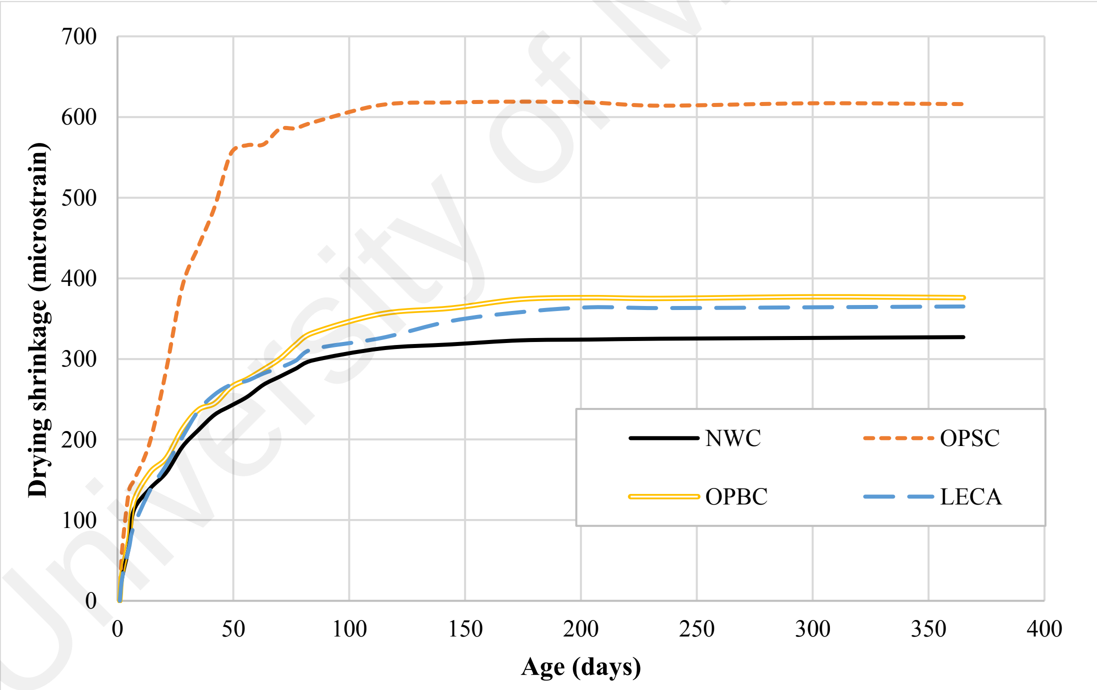


Figure 7.5: Development of drying shrinkage strains under 7-days moist curing

7.3.2 Effect of Curing Condition on the Drying Shrinkage

The curing conditions under which the concretes were allowed to cure are important considerations for the shrinkage. A prolonged moist curing period would allow better hydration of the cement and also reduces the shrinkage effect (Maslehuddin et al., 2013;

McKeen & Ledbetter, 1969). A concrete without curing will show higher early-age shrinkage and be prone to cracking. Therefore, it is important to keep moisture on concrete surface for longer than few hours to slow down the drying process of the concrete.

Table 7.3 shows the early-age drying shrinkage of all the prepared mixes. The OPSC, OPBC and NW concretes showed lower early-age drying shrinkage under 7-days curing compared to uncured concretes. The reduction on drying shrinkage due to curing in the mix OPSC was higher than OPBC and NWC mixes. The results also confirm that the curing time is essential to minimize the shrinkage at early ages for all concretes except LECA lightweight concrete. Oliveira et al. (2015) reported that curing causes a lower drying mass loss and causes a delaying effect in the development of shrinkage allowing the natural strength development. However, the mix LECA showed higher shrinkage value on average about 20% under 7-day moist curing compared to air drying. CCAA (2005) reported that the insufficient bleeding on surface of concrete will increase the early-age shrinkage and the tendency of the concrete to crack.

The long-term shrinkage strains of all the prepared mixes are shown in Table 7.4. It is found that the cured specimens showed higher drying shrinkage than the air-dried specimens. Neville (2008) and Mehta and Monteiro (2006) reported that after curing period, the concrete is subjected to low relative humidity, which resulted a driving force for moisture migration out of the material and causes volume reduction. Similarly, swelling arises once there is increase in moisture content due to the absorption of water. Therefore, due to the curing of the concrete, it resulted higher long-term drying shrinkage.

Table 7.3: Effect of curing on drying shrinkage of concretes at early age

Mix codes	Drying shrinkage (x10^{-6})						Compared average
	Uncured			7 days cured			
	Age (days)			Age (days)			
	3	7	14	3	7	14	%
NWC	45	121	160	29(-36%)	112(-7%)	139(-13%)	-19
OPSC	108	188	247	63(-42%)	149(-21%)	197(-20%)	-28
OPBC	38	140	182	35(-8%)	123(-12%)	159(-13%)	-11
LECA	21	81	122	28(+33%)	92(+14%)	137(+12%)	+20

(-) decrease in shrinkage due to 7 days curing, (+) increase in shrinkage due to 7 days curing

Table 7.4: Effect of curing on long term drying shrinkage of concretes

Mix codes	Drying shrinkage (x10^{-6})								Compared average
	Uncured)				7 days cured				
	Age (days)				Age (days)				
	28	90	180	365	28	90	180	365	%
NWC	208	284	317	319	191(-8%)	301(+6%)	324(+2%)	327(+3%)	+1
OPSC	410	536	558	567	390(-5%)	594(+11%)	619(+11%)	616(+9%)	+7
OPBC	244	335	359	361	213(-13%)	336(+0%)	375(+4%)	376(+4%)	-1
LECA	186	308	340	344	203(+9%)	315(+2%)	360(+6%)	365(+6%)	+6

(-) decrease in shrinkage due to 7 days curing, (+) increase in shrinkage due to 7 days curing

7.4 Comparison of Drying Shrinkage Strains with Prediction Model

The experimental results of the drying shrinkage strain were compared with the prediction model proposed by ACI209R (2008). The ACI209R (2008) model is predominately used in the USA, and has been incorporated into many of the building codes. This model was developed empirically and is based on drying shrinkage data obtained prior to 1968. This model equations can be used to predict the drying shrinkage of normal weight and all types of lightweight concretes. A detailed description of the method can be found in ACI209R (2008). The ACI-209R proposed shrinkage prediction model as, $S(\mathrm{t}, \mathrm{t_c})$ at time t (days) measured from start of drying at tc (days), and S_∞ is the ultimate shrinkage.

$$S_{(\mathrm{t,\,tc})} = \frac{(\mathrm{t} - t_c)}{f + (\mathrm{t} - t_c)} * S_\infty \qquad (7.1)$$

$$f = 26.0\, e^{\left[1.42*10^{-2}\left(\frac{V}{S}\right)\right]} \qquad (7.2)$$

$$S_\infty = 780 \times 10^{-6} \, (Y_{sh}) \tag{7.3}$$

$$Y_{sh} = K_1' * K_2' * K_3' * K_4' * K_5' * K_6' * K_7' \tag{7.4}$$

where, f is the compressive strength of the concrete in MPa. While to take into account the size and geometry on the drying shrinkage of concrete specimens, the $\left(\frac{V}{S}\right)$ is the volume to surface area ratio. Where, Y_{sh} represents the product of several factors that take into account. K_1' is the curing time coefficient; K_2' is the relative humidity coefficient; K_3' depends on volume to surface area ratio; K_4' is the slump factor; K_5' is the fine aggregate ratio (fine aggregate to the total aggregates); K_6' is the cement content in kg/m^3 and K_7' is the air content (%).

The average values were taken and development of shrinkage curve was plotted as can be seen in Figure 7.6. It was observed that the mix OPSC showed very close shrinkage results to the predicted results in early ages up to 14 days. However, in later ages, the difference between the results was consistently increased, after one year of drying it was about 30%. Whereas, the ACI-209R prediction model of shrinkage strain gave close results to shrinkage values of the mixes NWC, OPBC and LECA in both early and later ages. After 30 days, the mixes NWC, OPBC and LECA showed the difference between experimental and predicted results of about 7%, 27% and 1%, respectively, however, after one year of drying, these ratios were about 20%, 10% and 14%, respectively.

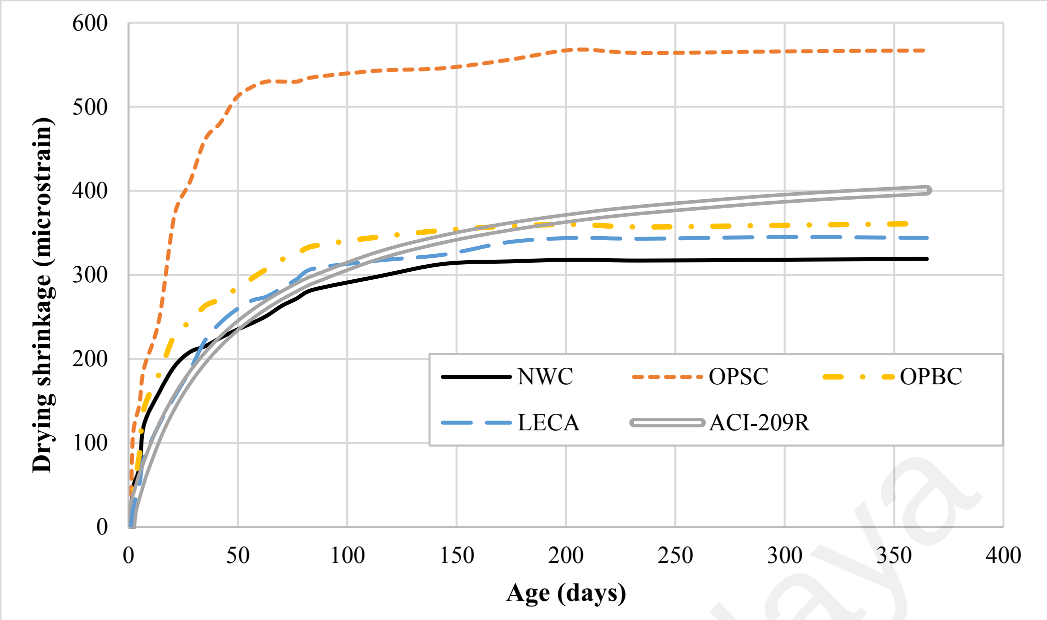


Figure 7.6: Development of drying shrinkage with average prediction model results under air-drying condition

Similar to air-dry condition, the ACI-209R model predicted almost similar results for all the mixes under 7-day moist curing. Therefore, the average values were taken and development of shrinkage curve was plotted as can be seen in Figure 7.7. It was observed that the mix OPSC showed very close shrinkage results to the predicted results in first week of drying. However, at later ages the difference between the results was constantly increased, after one year of drying it was about 46%. Whereas, the ACI-209R prediction model of shrinkage strain gave close results to shrinkage values of the mixes NWC, OPBC and LECA in both early and later ages. After 60 days, these mixes showed the average difference between experimental and predicted results of about 22%. However, with the increasing trend, the mixes OPBC and LECA showed the average difference between experimental and predicted results of about 16% and 2% after six months and one year, respectively. Whereas, this ratio for mix NWC was about 6% and 0%, at six months and one year, respectively. In general, it can be concluded that this model code of NWC is also suitable to predict drying shrinkage of OPBC and LECA concretes at short term and long term ages. However, for OPS concrete a new modified model can be developed to predict the drying shrinkage.

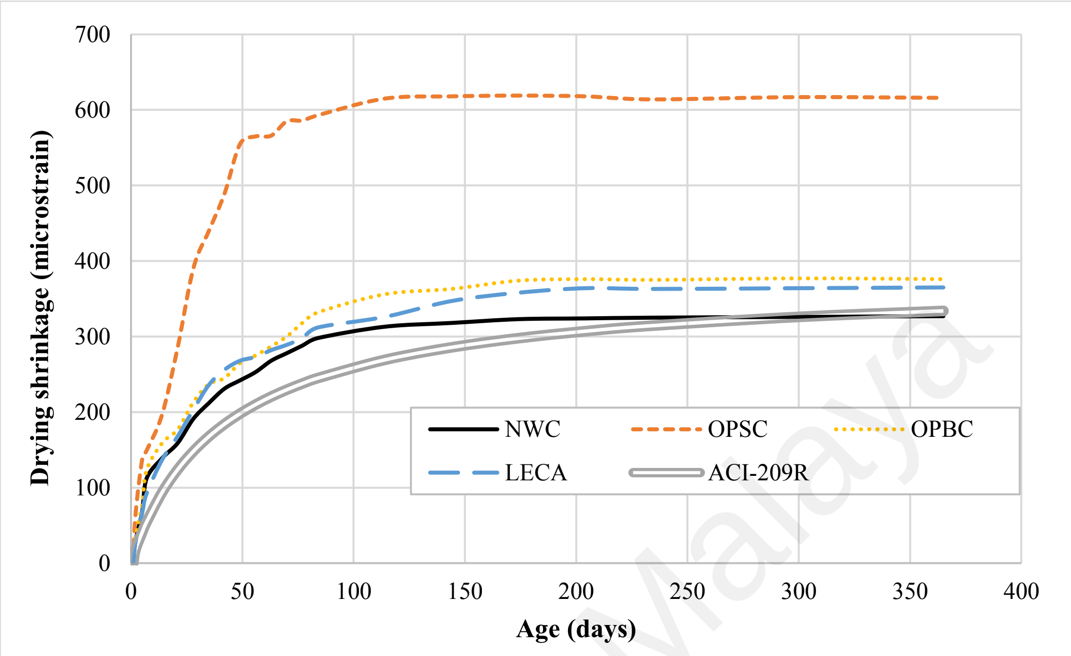


Figure 7.7: Development of drying shrinkage with average predicted results under 7-day moist cuing condition

7.5 Conclusions

The early-age and long-term drying shrinkage strain of the concretes containing different types of aggregates were discussed. Based on the test results, the following conclusions can be drawn:

1. In early ages, all the prepared lightweight concretes (OPSC, OPBCC and LECA concrete) showed closer results of drying shrinkage to the conventional concrete (NWC). Later at 56 days age, the OPSC showed significantly higher drying shrinkage than the other concretes. Whereas, the OPBCC and LECA concrete showed similar trend to the conventional concrete.
2. Long-term (after one year of drying), the OPS concrete on average showed 40% higher drying shrinkage strains compared to the other concretes.

3. Compared to OPSC, the aggregates in other concrete mixes (OPBC, LECA and NWC) showed better bonding with the cement paste in Interfacial Transaction Zone (ITZ) and reduced the formation of micro-cracks, which resulted significantly lower drying shrinkage strains.
4. OPSC showed significantly higher drying shrinkage results under 7-day moist curing, which is about 47%, 41% and 39% higher than the shrinkages of mixes NWC, LECA and OPBC, respectively. Whereas, the average difference between the shrinkage values of NWC and OPBCC and LECA concretes was found in the range of 10-13%.
5. The OPS, OPBC and conventional concrete shows lower early-age shrinkage under 7-days curing, and the reduction in OPSC was higher than OPBCC and NWC. This also confirm that the curing time is essential to minimize the shrinkage at early ages. Whereas, in long-term shrinkage, the cured specimens showed higher drying shrinkage than the uncured ones, and the average difference for OPSC and LECA concretes was found almost similar of about 6%.
6. The ACI drying shrinkage prediction model estimated lower results for the OPS concrete with the average difference of about 50%. However, for other concretes (mixes NWC, OPBC and LECA), the ratio between the experimental and the predicted results was found in the range of 20-30%.

CHAPTER 8: PREDICTION MODELS FOR DRYING SHRINKAGE IN PALM OIL BY-PRODUCT LIGHTWEIGHT CONCRETES

8.1 Introduction

Among civil engineering materials, concrete is the only material that undergoes an instantaneous deformation upon loading because of its hygroscopic nature and aging (ACI 209R, 2005). “Shrinkage” can be defined as the decrease in length or volume of hardened concrete with time, which is primarily due to moisture and chemical changes in concrete (ACI 209R, 1997). Among several types of shrinkage, drying shrinkage plays an extremely important role in the contraction of concrete because its driving force removes adsorbed water from hydrated cement paste. Drying shrinkage in concrete structures is critical particularly under severe climate conditions and seasonal fluctuations in temperature and humidity. Sudden and continuous variations in temperature and humidity create contraction/expansion cycles in concrete, which induces shrinkage and thermal stresses leading to microcracking in concrete members (Asad, 1995).

The development of microcracks caused by deformation due to time-dependent properties significantly reduces the safety margins, which majorly affects the service life and durability of concrete structures (Bazant, Kim, & Panula, 1991; Boucherit, Kenai, Kadri, & Khatib, 2014). The magnitude of shrinkage strains depends on several factors such as relative humidity, type and quantity of binder, specimen size, water-to-binder ratio, type and quantity of aggregates, curing conditions and mixing method. The complexity of the subject is that the combined effect of the mix design parameters on the equilibrium drying shrinkage of concrete is not well understood (Bazant, 2001; Young, 1988). Every increment in concrete shrinkage strain increases the deflection of a concrete member, affects cracking progress, and causes loss of prestress (Sakata, Tsubaki, Inoue, & Ayano, 2001). Moreover, the complexity of the phenomena including pressure of disjunction, capillary pressure, and surface tension and their effects on the properties of cementing materials makes the establishment of a theoretical model difficult, which takes into account all the factors influencing the drying shrinkage strain and their coupled effects (Boucherit et al., 2014). Thus, to check the safety, durability and serviceability of concrete structures, a means of predicting creep and shrinkage over the long term is highly important (Sakata et al., 2001).

The shrinkage of the concrete is known to basically occur due to the contraction or expansion of the cement paste in the mechanism, and this volumetric change can only be opposed by the aggregates. Structural lightweight aggregate concrete (SLWAC) is modified at these factors. This, SLWAC is expected to behave differently compared to the normal weight concrete (NWC) (Bogas et al., 2014). First, the less rigid porous aggregates decrease the restriction effect on the paste deformation (Aslam et al., 2016; Zhang et al., 2005). Second, the lightweight concrete (LWC) is usually characterized by larger volumes of better-quality paste and lower volume of coarse aggregates (Bogas, 2011; Holm & Bremner, 2000). Therefore, the long-term shrinkage of LWC should be higher than that of NWC of the same grade. Although, some studies (Shafigh et al., 2014; Wegen & Bijen, 1985) reported that the shrinkage of the lightweight aggregate concrete (LWAC) is less than or similar to the shrinkage of NWC, for general design work, the shrinkage of LWAC is suggested to be between 1.4 and 2 times that of NWC (Clarke, 2002).

Since the last few decades, the agricultural waste from the oil palm industry was successfully utilized as a lightweight aggregate to produce green and sustainable concretes. Previous studies (Aslam et al., 2016; Mannan et al., 2006; Shafigh et al., 2012) reported that the oil palm shell (OPS) or palm kernel shell (PKS) can be used as a lightweight aggregate (LWA) for producing structural LWAC with a density of 20-25%

lower than the NWC and with good mechanical properties and durability properties. However, this concrete has remarkably high drying shrinkage compared to artificial lightweight and normal weight concretes. Aslam et al. (2016) investigated the drying shrinkage of the OPS concrete by incorporating oil-palm-boiler clinker (OPBC) in the OPS concrete. They reported that the contribution of OPBC in OPS concrete along with the small reduction of water-to-cement significantly reduced the drying shrinkage compared to control OPS concrete. Although several studies on the drying shrinkage of the OPS and OPS-OPBC concretes are available in the literature, the prediction models of these types of concretes still need further investigations. Therefore, in this study, 10 different drying shrinkage prediction models were selected from the literature to compare the experimental and predicted results of the OPS and OPS-OPBC concretes.

8.2 Experimental Program

8.2.1 Materials and Mix Proportions

Ordinary Portland cement (OPC) of grade 48 conforming to Malaysian Standards (MS 522): Part 1: 2003 was selected as the cement in this investigation. The specific gravity and the Blaine specific surface area of the cement were 3.14 and 3510 cm^2/g, respectively. The drinking water was utilized for mixing and curing the concrete specimens, whereas the Sika ViscoCrete with a total chloride ion content of ≤ 0.1, which is compatible for all types of cements according to British Standards (BS-5075), was used as super-plasticizer (SP) to control the workability of the concrete.

The locally available agro-waste materials, such as OPS and OPBC, from the palm oil industry were used as coarse aggregates. Both aggregates were stored at the open atmosphere for approximately 6-8 months to properly dry them and remove the fibers from the OPS surface. Then, the OPS aggregates were washed in the concrete mixer to remove the oil and impurities from the surface, whereas, the OPBC aggregates were crushed using a crushing machine. The aggregates were sieved to achieve the same grading. Locally available mining sand with a maximum grain size and fineness modulus of 4.75 mm and 2.89, respectively, was used as a fine aggregate. Table 8.1 shows the physical properties of the fine and coarse aggregates.

Three different lightweight concrete mixes were designed by using OPS and OPBC lightweight aggregates. In all the mixes, the volume of aggregates, cement and SP contents were kept constant. The details of the designed concrete mixes and the properties of the concretes are shown in Table 8.2. The OPS concrete (mix P) was considered a control concrete containing only agro-waste material. However, in the remaining mixes, the OPS aggregate in mix P was partially replaced with inert OPBC material at 30% and 50% by volume. The main reason for the partial substitution of OPBC in OPS concretes was to identify the blended effect of these aggregates on the drying shrinkage strain as explained in other works (Aslam et al., 2016). However, in this study, the experimental results of the drying shrinkage were compared with the shrinkage prediction model results.

Table 8.1: Physical properties of the aggregates

Properties	Fine aggregate	Coarse aggregates	
		OPS	OPBC
Specific gravity	2.68	1.19	1.69
Density (compacted) (kg/m^3)	1657	610	860
24-h water absorption (%)	1.2	20.5	7.0

Table 8.2: Concrete mix proportions

Mix code	Cement (kg/m^3)	w/c ratio	SP (% cement)	Sand (kg/m^3)	Coarse aggregate (kg/m^3)		Slump (mm)	Density (kg/m^3)	28-day compressive strength (MPa)
					OPS	OPBC			
P	480	0.36	1	890	360	0	55	1790	36
C30	480	0.36	1	890	252	153	90	1833	43.5
C50	480	0.295	1	890	180	256	52	1940	51.5

8.2.2 Test Methods and Specimen Sizes

The concrete mixes were prepared in the rotating drum mixer. Initially, all the materials were weighed in dry condition, and the OPS and OPBC lightweight aggregates were pre-soaked in water for 24 hours to control the effective water content of the concrete. Then, the pre-soaked aggregates were air-dried in the laboratory environment for 2-3 hours to obtain saturated surface dry condition. For each mix proportion, the cement and aggregates were placed in the mixer and mixed for 2 minutes. Then, the mixture of 70% mixing water with SP was added to the mixer, and mixing was continued for another 3 minutes. After, the remaining water was added to the mixture, and mixing was continued for another 5 minutes. Then, the fresh properties of the concretes were measured (Table 8.2).

Two specimens with the size of 100 × 100 × 300 mm were prepared to measure the drying shrinkage strains of all the designed concrete mixes. After moulding, the specimens were covered with plastic sheets and stored in the laboratory atmosphere. After demoulding and curing, the concrete samples were placed in the uncontrolled room. The drying shrinkage strains were measured by demountable mechanical strain gauge (DEMEC) with a resolution of 0.001 mm. The DEMEC points were fixed 200 mm apart with V-tech 502 Super Glue on the three plain sides of the samples and placed in the uncontrolled room for 1-2 hours, so that the applied glue becomes properly dry. Then the initial readings were recorded by the equipment (gauge type PLR-60-11).

The specimens for the drying shrinkage of concretes were cured in water for 7 days. Then, the samples were placed in an uncontrolled room in the laboratory. However, for 28-day compressive strength, the 100 mm cube specimens were placed in water until the age of testing. The curing water has an average temperature of 22 ± 2^{o}C, whereas the uncontrolled room in the laboratory has a temperature of 30 ± 2°C and a relative humidity of 74 ± 4%.

8.3 Results and Discussions

8.3.1 Shrinkage Prediction Models

In the previous century, many theories were presented after the first observation of the drying shrinkage strain in concrete (Boucherit et al., 2014). Based on those theories, several mathematical prediction models were proposed to estimate the shrinkage strains, and some of those models were also included to the codes of practice to ease the structural analysis. Presently, several models are available to predict the long-term drying shrinkage strain of the concrete. Two major practical considerations were carried out in predicting the models for shrinkage, namely, mathematical form of their time dependency, and fitting of the parameters and resulting expressions. Ten drying shrinkage prediction models were selected from the standards, and researchers investigated the drying shrinkage strain of the prepared OPS and OPS-OPBC lightweight concretes. The selected set of values for the prediction models are mentioned in Table 8.3.

Table 8.3: Selected parameters for prediction models

Factors / Items	Input data
Start of drying	7 days
Relative humidity (%)	76%
28-day compressive strength (MPa)	36-52
Cement type	Normal OPC
Size and shape of specimen (mm)	100*100*300

8.3.1.1 ACI-209R shrinkage model

The ACI 209R (2008) model was proposed and predominately used in the USA and has been included in several building codes. This method was developed empirically and was based on the drying shrinkage strain results achieved prior to 1970. The ACI 209R model can be used to predict the drying shrinkage strains of normal weight and lightweight concretes with low to moderate strength, under controlled exposure conditions. The expressions are empirically based and do not model shrinkage phenomena. This document does not provide any coefficient to penalize the long-term shrinkage of LWC. The ACI-209R proposed shrinkage prediction model as, $S(t, t_c)$ at time "t" (days) measured from the start of drying at "t_c" (days), and S_∞ is the ultimate shrinkage, as can be seen in Equations 8.1 to 8.4.

$$S_{(t,\, tc)} = \frac{(t - t_c)}{f + (t - t_c)} * S_\infty \tag{8.1}$$

$$f = 26.0 * e^{\left[1.42 * 10^{-2}\left(\frac{V}{S}\right)\right]} \tag{8.2}$$

$$S_\infty = 780 \text{ x } 10^{-6} * (Y_{sh}) \tag{8.3}$$

$$Y_{sh} = K_1' * K_2' * K_3' * K_4' * K_5' * K_6' * K_7' \tag{8.4}$$

where, f = 35 for concrete water-cured for 7 days to take into account the size and geometry on the drying of concrete specimens and $\left(\frac{V}{S}\right)$ is the volume to surface area ratio. The Y_{sh} represents the product of several factors that take into account. K_1' is the curing time coefficient, K_2' is the relative humidity coefficient, K_3' depends on volume to surface area ratio, K_4' is the slump factor, K_5' is the fine aggregate ratio (fine aggregate to the total aggregates), K_6' is the cement content in kg/m^3 and K_7' is the air content (%). As can be seen in Table 8.4, the ACI209R (2008) is only the model which covers almost all types of factors that has a direct or indirect effect on the drying shrinkage of the concrete.

Table 8.4: Selected factors for the prediction of drying shrinkage

Factors	Drying shrinkage prediction models									
	ACI-209R	EN1992	MC2010	CEB/FIP	AASHTO	GL2000	AS3600	JSCE	B3	SAK
Compressive strength (MPa)	*	*	*	*	*	*	*	*	*	*
Curing factor	*	-	-	-	*	-	-	*	*	-
Relative humidity	*	*	*	*	*	*	-	*	*	*
Volume to surface area ratio (v/s)	*	-	-	*	*	*	-	-	*	*
Slump factor	*	-	-	-	-	-	-	-	-	-
Fine agg. to total aggregate (A_f/A)	*	-	-	-	-	-	-	*	-	-
Cement type factor	*	*	*	*	-	*	-	*	*	*
Water	-	-	-	-	-	-	-	-	-	*
Air content factor	*	-	-	-	-	-	-	-	-	-
Cross-section	-	*	*	*	*	*	*	*	*	*
Environmental	-	-	-	-	*	-	*	*	-	-
Lightweight concrete factor	-	*	*	-	-	-	-	-	-	-

The development of the drying shrinkage strain and the ACI predicted value trend are shown in Figure 8.1. Based on the predicted values, the ACI model estimated almost the same results for all the prepared concrete mixes. However, each mix has different properties and was applied in the prediction model. Therefore, the average values were taken and plotted as shown in figure 8.1. After the first week of drying, the predicted results were found similar to the experimental results. By contrast, as the drying period increases, the predicted model under-estimated the values. After one year of drying, the ACI model predicted values were approximately 47%, 37%, and 18% lower than the experimental values for the mixes P, C30, and C50, respectively.

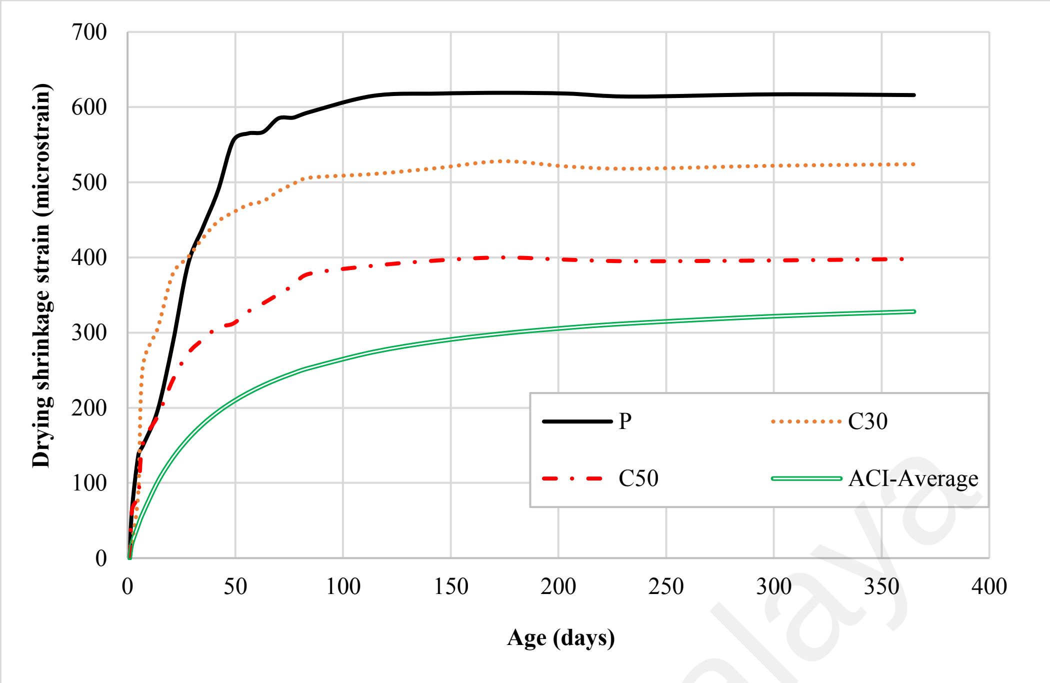


Figure 8.1: Development of the drying shrinkage (Experimental vs. average ACI209R (2008) prediction)

8.3.1.2 Eurocode shrinkage model

The standard document of European Standards (EN1992, 2010) reported that the shrinkage of LWC can roughly be estimated from the expressions defined for the NWC. The final drying shrinkage of LWC is multiplied by an empirical factor of 1.2. The EN1992 (2010) model considers the total shrinkage, which is divided into drying shrinkage and autogenous shrinkage, as can be seen in Equations 8.5 to 8.9. This model assumes that the autogenous shrinkage of LWC can be much smaller than that of NWC if the LWA is pre-soaked, but no suggestion is given for its estimation. Therefore, drying shrinkage for lightweight concretes was the only focus of this investigation.

$$\varepsilon_{cs} = \varepsilon_{cd} + \varepsilon_{ca} \tag{8.5}$$

where, ε_{cs} is the total shrinkage and ε_{cd} and ε_{ca} represent the drying shrinkage and the autogenous shrinkage, respectively. The drying shrinkage prediction model is expressed in Equations. 8.6 to 8.9.

$$\varepsilon_{cd} = \beta_{ds}(t, t_s).k_h.\varepsilon_{cd,0} \tag{8.6}$$

$$\varepsilon_{cd,0} = 0.85.\beta_{RH}.\left[(220 + 110.\alpha_{ds1}).\exp(\frac{-\alpha_{ds2}\cdot f_{ck}}{10})\right] \tag{8.7}$$

$$\beta_{ds}(t, t_s) = \frac{(t - t_s)}{(t - t_s) + 0.04\sqrt{h_0^3}} \tag{8.8}$$

$$\varepsilon_{cs}(t, t_s) = \eta_3 * \varepsilon_{cd} \tag{8.9}$$

where, k_h is a coefficient depending on the notional size h_0 and the factor $\beta_{as}(t)$ is given as a function of time t (in days). The factor $\beta_{ds}(t, t_s)$ takes into account the age of the observed specimen. Here t (in days) is for the age of the concrete which is being considered, t_s (in days) is the age of the concrete when it was first exposed to drying and h_0 is the notional size as before. The recommended parameters for the drying shrinkage prediction were mentioned in Table 8.4. Meanwhile, the details of the factors were considered as per suggestions and guidelines by EN1992 (2010).

The development of the drying shrinkage strain and the theoretical results of EN1992 are shown in Figure 8.2. In early ages, all the mixes showed similar experimental results with the predicted shrinkage values. However, at later ages after one year, the average difference between experimental and predicted shrinkage strain results for the mixes P, C30, and C50 were approximately 40%, 29%, and 7%, respectively. The average predicted values were similar to the experimental values of C50 concrete mix with an average difference in the range of approximately 6% - 8%.

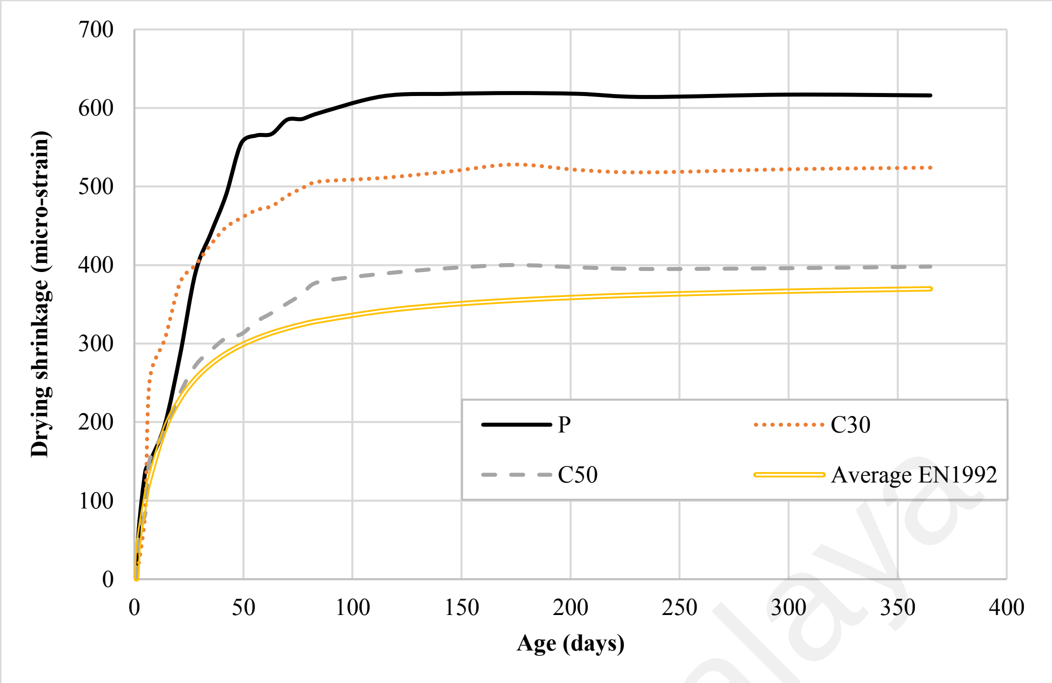


Figure 8.2: Development of drying shrinkage (Experimental and average EN1992 (2010) prediction)

8.3.1.3 Model Code 2010 shrinkage model

Similar to EN1992 (2010), the Model Code (MC) 2010 (2011) also reported that the shrinkage of LWC can be estimated from the equations proposed for the conventional concrete by multiplying an empirical factor of 1.2. This model also focuses on the total shrinkage, which contains both drying and autogenous shrinkages. However, the autogenous shrinkage of LWC is assumed to be much smaller than that of NWC if the LWA is pre-soaked, but no suggestion is given for its estimation. The detailed equations proposed by the MC 2010 (2011) model for the total shrinkage actually considered drying shrinkage and autogenous shrinkage, as shown in Equations 8.10 to 8.14.

$$\varepsilon_{cs}(t, ts) = \varepsilon_{cd} + \varepsilon_{ca} \tag{8.10}$$

where, $\varepsilon_{cs}\ (t, ts)$ is the total shrinkage strain between the age (t) and beginning of drying (t_s); and ε_{cd} and ε_{ca} represent the drying shrinkage and the autogenous shrinkage accordingly. The recommended parameters for the drying shrinkage prediction were mentioned in Table 8.4. The drying shrinkage is expressed as follows;

$$\varepsilon_{cd}(t) = \beta_{ds}(t, t_s).\beta_{RH}.\varepsilon_{cd,0} \quad (8.11)$$

$$\varepsilon_{cd,0} = \left[(220 + 110.\alpha_{ds1}).\exp(\frac{-\alpha_{ds2} \cdot f_{ck}}{10})\right] \quad (8.12)$$

$$\beta_{ds}(t, t_s) = \left[\frac{(t - t_s)}{(t - t_s) + 0.035\, h_0^2}\right]^{0.5} \quad (8.13)$$

$$\varepsilon_{cs}(t, t_s) = \eta * \varepsilon_{cd} \quad (8.14)$$

where, k_h is a coefficient depending on the notional size h_0. The factor $\beta_{ds}(t, t_s)$ takes into account the age of the observed specimen. Here t (in days) is for the age of the concrete which is being considered, t_s (in days) is the age of the concrete when it was first exposed to drying and h_0 is the notional size as before. The autogenous part of the shrinkage is defined as follows by the standard was calculated from the compressive strength. And the factor $\beta_{as}(t)$ is given as a function of time t (in days).

The development of the drying shrinkage strain and the theoretical results by MC 2010 are shown in Figure 8.3. The MC 2010 model predicted highly similar results for all the concrete mixes. Therefore, the average values were selected and plotted as shown in figure 8.3. In early ages up to 14 days, the predicted results of shrinkage were almost similar to the experimental results. However, at later ages after year of drying, the ratio between the experimental and predicted results was higher, which was approximately 34% for mix P and 22.5% for mix C30, whereas the mix C50 showed almost similar experimental results to the predicted results.

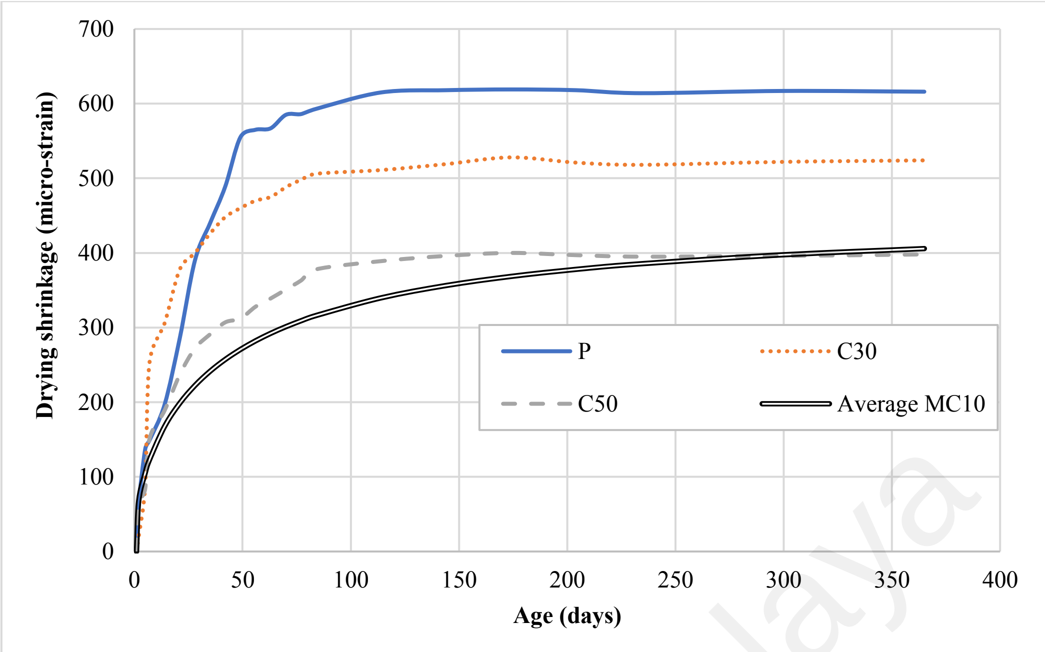


Figure 8.3: Development of drying shrinkage (Experimental and average MC2010 (2011) prediction)

8.3.1.4 CEB/FIP90 shrinkage model

Europeans typically use the prediction method developed in 1990 by the Comité Euro-International du Béton (CEB). The CEB/FIP90 (2001) model was derived using mathematical functions instead of tables and figures and has been optimized from information from a data bank of structural concrete. This model predicts the time dependent deformation of ordinary normal-weight concrete exposed to a temperature range of 5 °C and 30 °C and a relative humidity of 40% to 100%. The CEB/FIP90 model proposed Equations 8.15 to 8.18 of the drying shrinkage model which are as follows:

$$\varepsilon_s(t - to) = \varepsilon_{so} * \beta_s(t - to) \tag{8.15}$$

$$\varepsilon_{so} = [160 + 10.\beta_{sc}\left(9 - \frac{f_c'}{10}\right).\beta_{RM} \tag{8.16}$$

$$\beta_s(t, t_o) = \left[\frac{(t - t_o)}{\beta_{SH} + (t - t_o)}\right]^{0.5} \tag{8.17}$$

$$\beta_{SH} = 350 * \left[\frac{2 * {A_c}/{\mu}}{100}\right]^2 \qquad (8.18)$$

where; ε_s is drying shrinkage of the concrete, t is the age of concrete in days, to is age of concrete at beginning of drying in days, ε_{so} is the notional shrinkage coefficient, β_{sc} is the cement type factor, f_c' is the 28-day compressive strength, h is the relative humidity in decimals, A_c is the cross-sectional area in (mm^2), and μ is the exposed perimeter (mm) of the sample. The recommended parameters for the drying shrinkage prediction were mentioned in Table 8.4.

The development of the drying shrinkage strain and the theoretical results by CEB/FIP90 model are shown in Figure 8.4. The CEB/FIP90 model predicted highly similar results for all the concrete mixes. Therefore, the average values were selected and plotted as shown in figure 8.4. In early ages up to 21 days, the predicted results of shrinkage were almost similar to the experimental results. However, at later ages after year of drying, the ratio between the experimental and predicted results was higher, which was approximately 45% for mix P, 35% for mix C30, and 14% for mix C50.

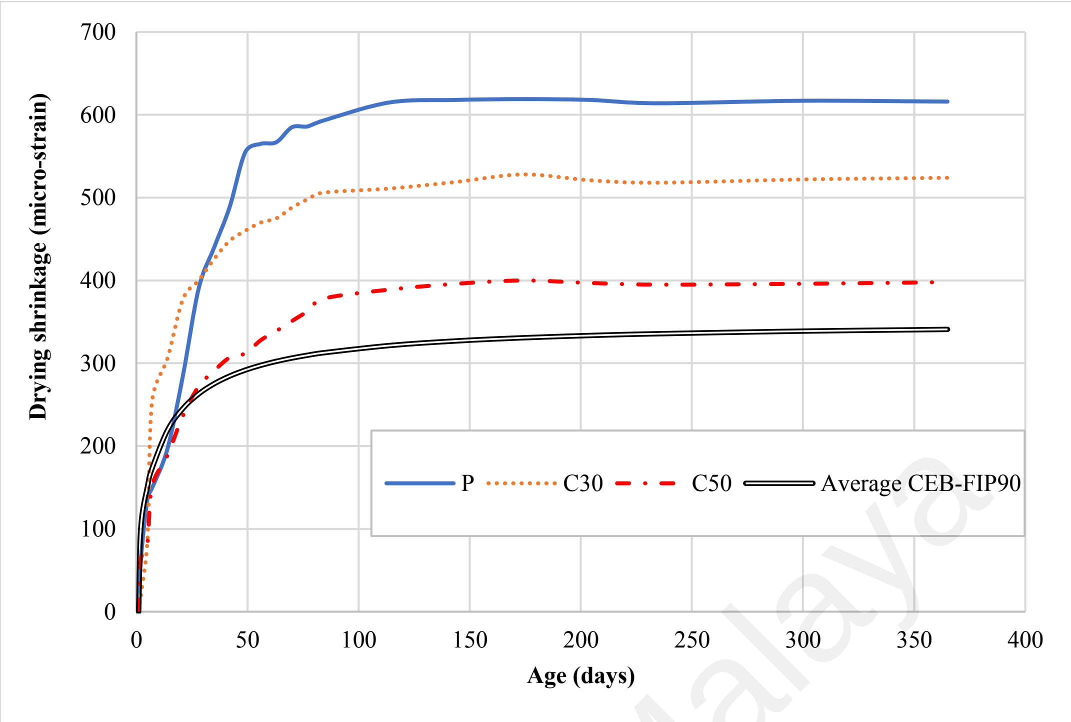


Figure 8.4: Development of drying shrinkage (Experimental and average CEB/FIP90 (2001) prediction)

8.3.1.5 AASHTO-LRFD shrinkage model as modified by Shams and Kahn (2000)

AASHTO-LRFD (2004) general expression for shrinkage is similar to the ACI209R (2008) method including the values for f as 35 and 55 for moist and steam-cured concretes, respectively. The expression for calculating ultimate shrinkage is different from ACI expression and contains slightly different correction factors. Shams and Kahn (2000) proposed some changes to AASHTO-LRFD creep and shrinkage expression to better predict long-term strains of the concretes.

$$\varepsilon_{sh(t-t_0)} = \varepsilon_{sh\infty} * k_{vs} * k'_H * k'_{t0} * \left[\frac{(t - t_0)}{f + (t - t_0)}\right]^{0.5} \tag{8.19}$$

where, $\varepsilon_{sh(t-t_0)}$ is the drying shrinkage strain, $\varepsilon_{sh\infty}$ is the ultimate shrinkage strain, k_{vs} is the size factor, k'_H is the ambient relative humidity factor, k'_{t0} is factor for maturity at the beginning of drying and f is the compressive strength of concrete. The recommended parameters for the drying shrinkage prediction were mentioned in Table 8.4.

The development of the drying shrinkage strain and the theoretical results are shown in Figure 8.5. As shown in the figure, in the first month of drying, the predicted results of shrinkage were found similar to the experimental results. However, at later ages after year of drying, the model predicted better results as it underestimated the predicted results for mixes P and C30 of approximately 28% and 15%, respectively, compared to the experimental shrinkage results. However, for the C50 mix, the estimations were approximately 11% higher than the experimental results.

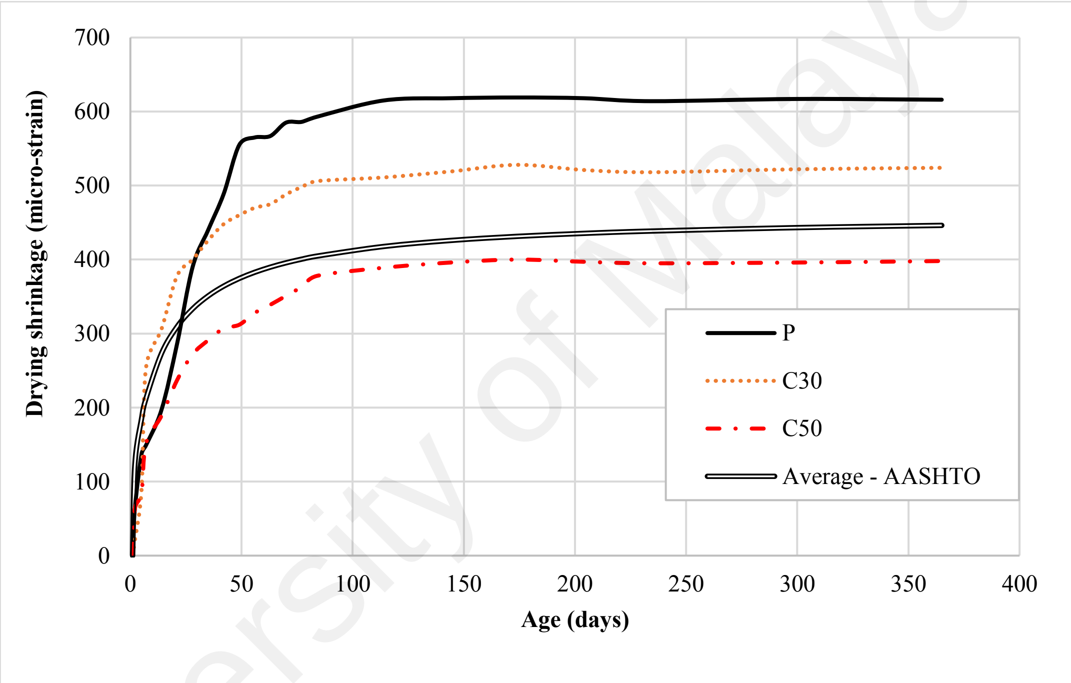


Figure 8.5: Development of drying shrinkage (Experimental and average AASHTO-LRFD (Shams & Kahn, 2000) prediction)

8.3.1.6 Gardner and Lockman (GL2000) shrinkage model

Gardner and Lockman (2001) proposed the GL2000 model to predict the shrinkage strains of the concrete. This model is effective in predicting the shrinkage of normal-strength concretes with a 28-day compressive strength of less than 82 MPa and water-to-cement ratio ranging from 0.4 to 0.6. They reported that this model can be used to accurately predict the shrinkage regardless of which mineral by-products, admixtures, casting temperature, or curing regime is considered. However, this model assumes that the shrinkage strains reduce with the increase of compressive strength and modulus elasticity. The detailed expressions of this model for the prediction of drying shrinkage strain are explained below.

$$\varepsilon_{sh}(t, t_0) = \varepsilon_{shu} * (1 - 1.18 * h^4) * \left[\frac{(t-t_0)}{(t-t_0)+0.15*({}^{V}/_{S})^2}\right]^{1/2} \tag{8.20}$$

$$\varepsilon_{shu} = 1000 * K * \left(\frac{30}{f_c'}\right)^{\frac{1}{2}} * 10^{-6} \tag{8.21}$$

where, ε_{sh} is the shrinkage strain, ε_{shu} is the ultimate shrinkage strain, f_c' is 28-day compressive strength (MPa), h is the relative humidity in decimals, V is the specimen volume (mm^3) and S is the surface area of the specimen surface (mm^2). The recommended parameters for the drying shrinkage prediction were mentioned in Table 8.4.

The development of the drying shrinkage strain and the theoretical results are shown in Figure 8.6. As shown in the figure, in the first 30 days of drying, the predicted results of shrinkage were found similar to the experimental results. Therefore, the average values were selected from the predicted results and are plotted in figure 8.6. In the first 7 days of drying, the predicted results of shrinkage were found similar to the experimental results for all the mixes, whereas, the similar values were achieved up to 21 days for mixes P and C50. However, the model predicted the same trend of shrinkages for C50 concrete up to 120 days of drying. Then the theoretical values were overestimated and the trend steadily increased. At later ages after year of drying, the model predicted better results as it underestimated the predicted results for mixes P and C30 by approximately 25% and 11%, respectively, compared to the experimental shrinkage results. However, for the mix C50, the estimations were approximately 14% higher than the experimental results.

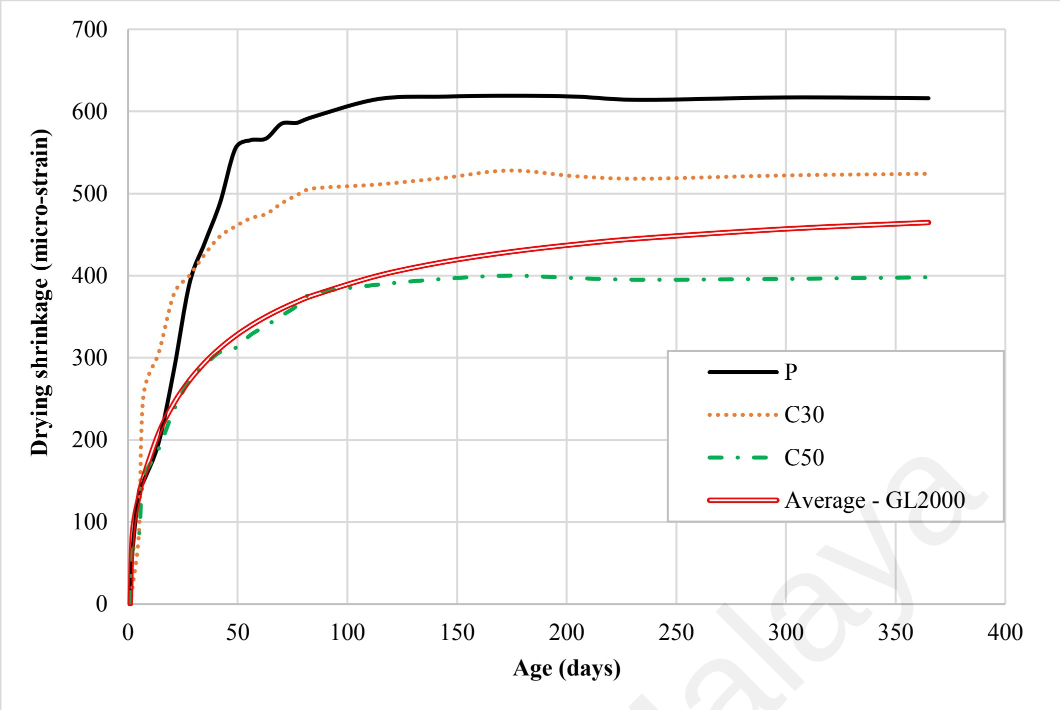


Figure 8.6: Development of drying shrinkage (Experimental and average GL2000 predictions)

8.3.1.7 Australian Standards model for shrinkage prediction

Australian Standards (AS) 3600 (2001) model reported that the design shrinkage for the normal-strength concrete (≤ 50 MPa) can be calculated at any time after the beginning of drying by using some basic equations. However, the proposed method in the AS 3600 that aims to predict the shrinkage strain was revised and extended to include high-strength concrete (HSC) where the causes, magnitudes, mechanisms, and rates of the development of shrinkage are significantly different than the normal-strength concrete. Gilbert (1998) proposed the modified version of shrinkage prediction model for normal- and high-strength concretes. This model considers the total shrinkage strain (ε_{cs}) and was divided into two components, endogenous shrinkage (ε_{cse}) and drying shrinkage (ε_{csd}). The detailed specifications of this model can be found from the original source (Gilbert, 2002). Moreover, the important expressions and equations of this model are shown below.

$$\varepsilon_{cs} = \varepsilon_{cse} + \varepsilon_{csd} \tag{8.22}$$

where, ε_{cs} is the total shrinkage strain, $\varepsilon_{cse} + \varepsilon_{csd}$ are the endogenous and drying shrinkages, respectively. Whereas, the drying shrinkage at any time "*t*" (in days) after the commencement of drying can be computed by using the expression as;

$$\varepsilon_{csd} = k_1 * k_4 * \varepsilon_{csd,b} \tag{8.23}$$

The basic drying shrinkage $\varepsilon_{csd,b}$ is given by:

$$\varepsilon_{csd,b} = (1.0 - 0.008 f_c') \, \varepsilon^*_{csd,b} \tag{8.24}$$

where, *f*c' is 28-day compressive strength (MPa), $\varepsilon^*_{csd,b}$ is the shrinkage of the aggregates depends on the quality of the local aggregates and may be taken as $800*10^{-6}$ for Sydney and Brisbane, $900*10^{-6}$ for Melbourne and $1000*10^{-6}$ elsewhere. The recommended parameters for the drying shrinkage prediction were mentioned in Table 8.4.

The relationship between the experimental and predicted drying shrinkage results with the respective age is shown in Figure 8.7. For mix P, the predicted results were almost similar to the experimental values in the first 30 days. Consequently, the prediction model underestimated the shrinkage results up to 10 months. Then the model predicted similar results with an average difference of approximately 6%, whereas, the AS3600 model predicted almost the same trend of the shrinkage results for the mix C30 with an accuracy of approximately 100%. For mix C50, in the early ages, the predicted results were found similar to the experimental values. However, at later ages, the results were overestimated with an average difference of approximately 17% after one year of drying.

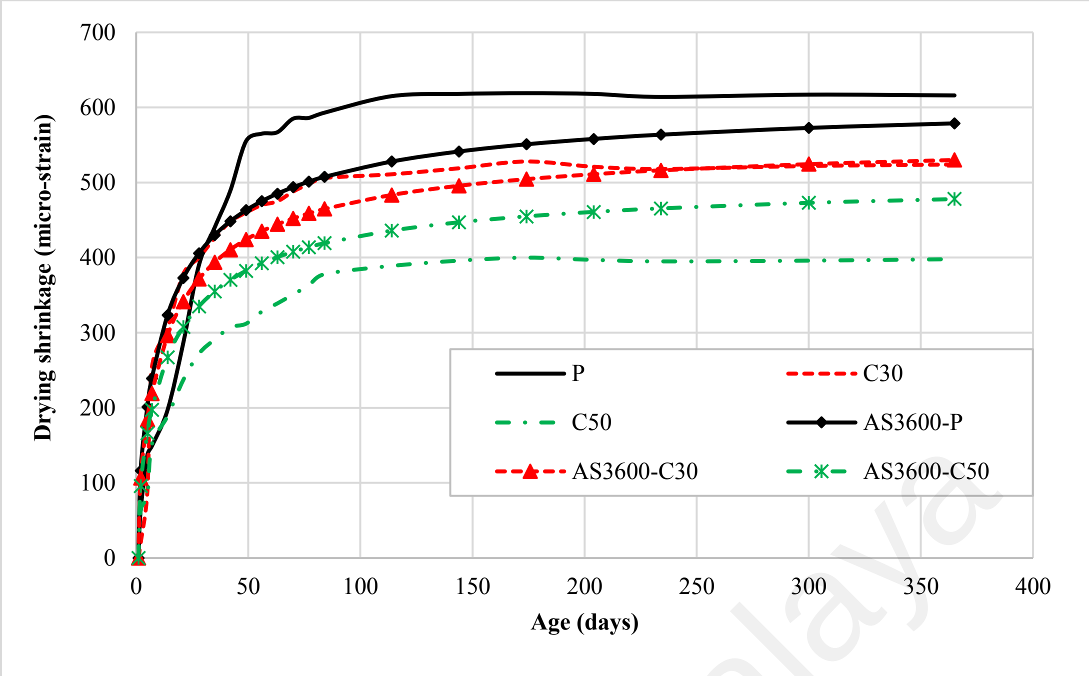


Figure 8.7: Development of drying shrinkage (Experimental and AS3600 model predictions)

8.3.1.8 Japan Standards 3600 shrinkage prediction model

Japan Standards (JSCE, 2007) described that the shrinkages of concrete were affected by the properties of the cement and aggregate, compaction and curing of concretes, temperature and relative humidity around structures, shape of the structural member, and the mix proportions of the concrete. Generally, the shrinkage strain of concrete used for confirmation purposes must be determined on the basis of the experimental data. However, if no such data are available, then the shrinkage strains can be calculated by using the following equations.

This code further explained that the relative humidity and size of the member strongly affect the rate and the magnitude of the shrinkage strain, which can normally be estimated using equation 25 for NWC with the compressive strength range of about 55 - 70 MPa. The recommended parameters for the drying shrinkage prediction were mentioned in Table 8.4. The detailed equations proposed by this standard are as follows:

$$\varepsilon'_{cs}(t, t0) = [1 - \exp\{-0.018(t - t0)^{0.56}\}] * \varepsilon'_{sh} \qquad (8.25)$$

$$\varepsilon'_{sh} = -50 + 78\left[1 - \exp\left(\frac{RH}{100}\right)\right] + 38log_eW - 5[log_e(\frac{V/_S}{10})]^2 \qquad (8.26)$$

where, $\varepsilon'_{cs}(t, t0)$ and ε'_{sh} are the shrinkage strain and the final value of shrinkage strains, respectively. *RH* is the relative humidity (%), *W* is the water content (kg/m^3) and *V/S* is the volume to surface area ratio.

The development of the drying shrinkage strain and the theoretical predicted results with the respective age are shown in Figure 8.8. The prediction model estimated almost similar shrinkage results for all concrete mixes. Therefore, the average values were selected from the predicted results and plotted in figure 8.8. As shown in the figure, in early ages (7 days), the prediction model estimated similar results with the experimental values for all the mixes. However, at later ages the prediction values showed a significantly increasing trend. After one year of drying, for mix P, the prediction model showed approximately 11% lower drying shrinkage results compared to the experimental achievements. However, this model overestimated the shrinkage strain for the OPBC-OPS mixes (mixes C30 and C50) which was approximately 4% and 27%, respectively.

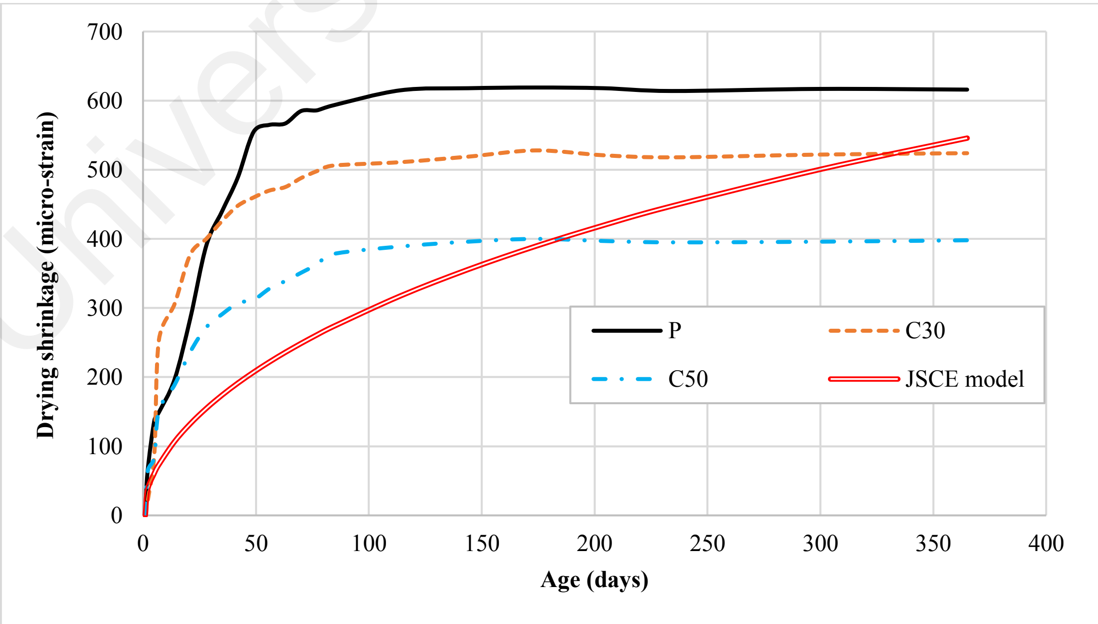


Figure 8.8: Development of drying shrinkage (Experimental and average JSCE3600 model predictions)

8.3.1.9 Sakata shrinkage prediction model

Sakata et al. (2001) developed an exponential model (SAK) for the drying shrinkage strain. The presented model was based on several parameters such as member geometry, relative humidity, cement, and water contents. The prediction model works for a wide range of compressive strengths. The main source of the date used in establishing the new prediction equations was the shrinkage data collected by RILEM and JSCE. The RILEM database holds data consisting of 419 shrinkage records (RILEM, 1999), whereas, the JSCE database contains 219 shrinkage records. The RILEM database consists mainly of data from the western countries, whereas the JSCE has collected data from papers published by Japanese organizations. The proposed prediction equations for the drying shrinkage strain are as follows:

$$\varepsilon_{sh}(t,t_0) = \frac{\varepsilon_{sh\infty} * (t-t_0)}{\beta+(t-t_0)} \tag{8.27}$$

$$\varepsilon_{sh\infty} = \frac{\varepsilon_{sh\rho}}{1+\eta * t_0} \tag{8.28}$$

$$\varepsilon_{sh\rho} = \frac{\alpha\,(1-h)W}{1+150\exp\left\{-\frac{500}{f_c'(28)}\right\}} \tag{8.29}$$

where, $\varepsilon_{sh}(t,t_0)$ is the drying shrinkage strain, t, t_0 is the current age and age at drying (days), $\varepsilon_{sh\infty}$ is the ultimate shrinkage strain, $f_c'(28)$ is the 28-day compressive strength (MPa), h is the relative humidity of the atmosphere, W is the unit water content (kg/m^3) and α is the factor for type of cement. The recommended parameters for the drying shrinkage prediction were mentioned in Table 8.4.

The relationship of the experimental and predicted drying shrinkage results with the respective age are shown in Figure 8.9. The prediction model estimated almost similar shrinkage results for all concrete mixes. Therefore, the average values were selected from the predicted results and plotted in figure 8.9. The SAK model estimated similar early age results with the experimental values for all the mixes. However, the model showed highly similar strong relationship with the experimental results of the C50 concrete mixture, whereas the predicted results were on average approximately 35% and 21% lower than the experimental shrinkage results for mixes P and C30, respectively.

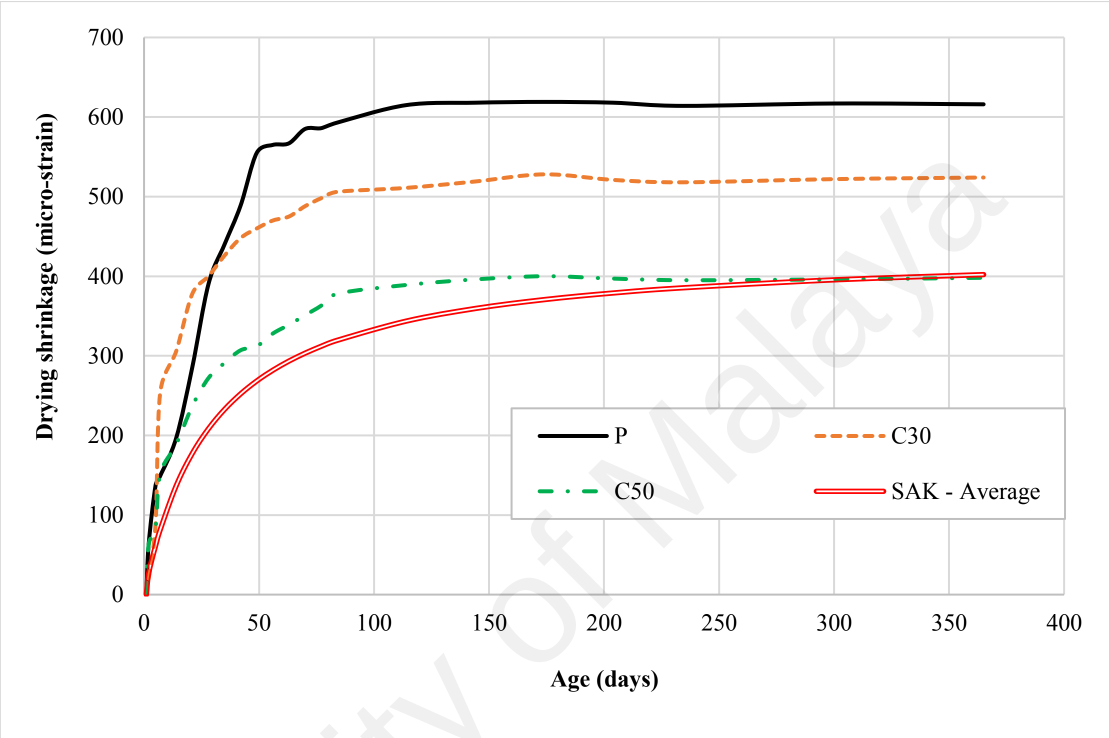


Figure 8.9: Development of drying shrinkage (Experimental and average SAK model predictions)

8.3.1.10 Bazant and Baweja shrinkage model

The B3 model developed by Bazant and Baweja is a better theoretically justified model than other proposed models (Bazant & Baweja, 2000; Recommendation & DE LA RILEM, 1995). The model is based on "a systematic theoretical formulation of the basic physical phenomena involved" and was "calibrated by a computerized data bank comprising practically all the relevant test data obtained in various laboratories throughout the world". Bazant and Baweja (2000) stated that the coefficient of variations for the B3 model is much lower than the ACI209R model and EN1992 model. The detailed expressions of this model are shown as follows:

$$\varepsilon_{sh}(t,t_0) = -\varepsilon_{sh\infty} * k_h * \tan h \sqrt{\frac{t-t_0}{\tau_{sh}}} \tag{8.30}$$

$$\varepsilon_{sh\infty} = -\alpha_1 * \alpha_2 * [0.00856 * w^{2.1} * (145 * f_c')^{-0.28} + 270] * \frac{(\frac{607}{4+0.85*607})^{1/2}}{(\frac{t_0+\tau_{sh}}{4+0.85*(t_0+\tau_{sh})})^{1/2}} \tag{8.31}$$

where, ε_{sh} is the drying shrinkage strain, $\varepsilon_{sh\infty}$ is the ultimate shrinkage strain, t is the age of concrete (days), t_0 is the age of concrete at the beginning of drying (days), α_1 is the cement type factor, α_2 is the curing factor, w is the water content (kg/m^3), f_c' is the 28-day compressive strength (MPa), k_h is the humidity dependent factor and τ_{sh} is the size and shape dependent factor. The recommended parameters for the drying shrinkage prediction were mentioned in Table 8.4.

The average prediction values were selected and compared with the experimental drying shrinkage results with the respective ages, as shown in Figure 8.10. This model predicted almost similar drying shrinkage results for all the mixes. In early ages (7 days), the prediction model showed almost similar results to the experimental values for the mixes. However, in the later ages, the model continuously underestimated the shrinkage results. After one year of drying, the predicted results were approximately 77%, 68%, and 58% lower drying shrinkage values compared to the experimental results for mixes P, C30, and C50, respectively.

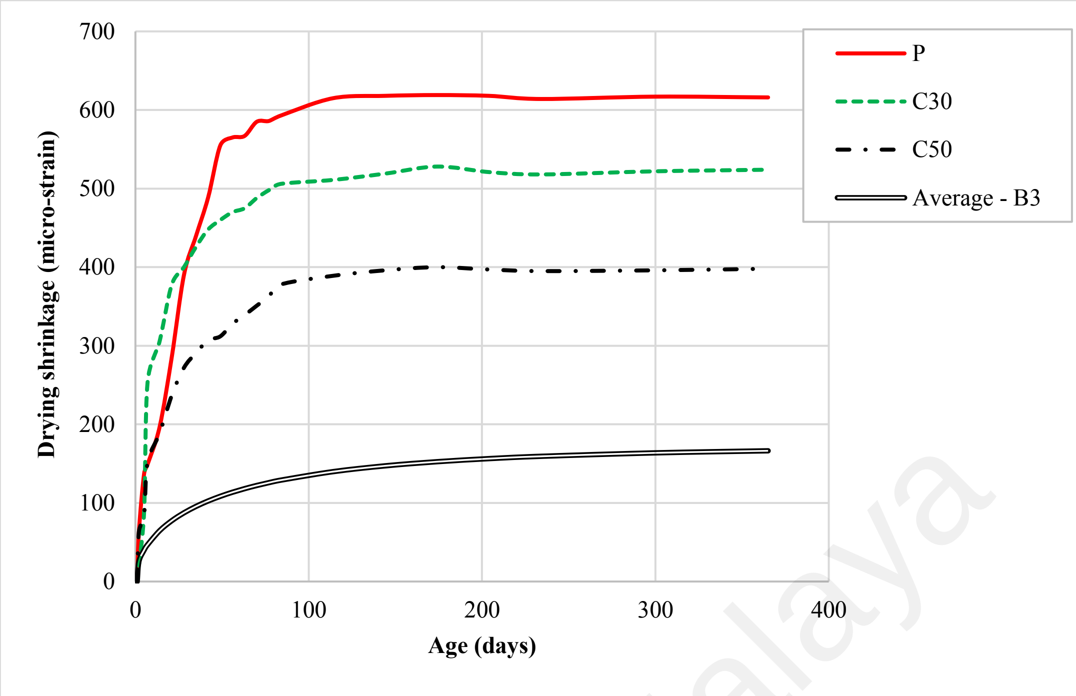


Figure 8.10: Development of drying shrinkage (Experimental and average B3 model predictions)

8.3.2 Accuracy of the Prediction Models

The development of the drying shrinkage strains for the OPS and OPS-OPBC concrete mixes (mixes P, C30, and C50) is compared with 10 prediction models (ACI209R, EN1992, MC2010, CEB/FIP1990, AASHTO-LRFD, GL2000, AS3600, JSCE3600, SAK and B3) as shown in Figure 8.11, Figure 8.12, and Figure 8.13. As can be clearly seen in the figures, in early ages, almost all the prediction equations estimated similar values to the experimental results. However, at later ages, only few models showed similar results. Therefore, to observe the accuracy of the prediction models for the prepared concrete mixes, two analyses were performed: an error percentage analysis (EP) and coefficient of variation (CV). For EP analysis, the residual values were calculated at the early age (14 days) and the long-term age (365 days) by considering the respective equations (Eq. 8.32).

$$\text{Residual value} = \text{Predicted shrinkage value} - \text{Experimental shrinkage value} \qquad (8.32)$$

By contrast, the positive residual values indicate that the model overestimates the experimental data, and the negative residual values specifies that the model underestimates the experimental data (Khanzadeh, 2010). In the EP analysis, the model with the minimum average error percentage can be considered the best predictor and the best model for the CV test and thus is the model with the lowest value. The error percentage analysis and the coefficient of variation were calculated as follows:

Error Percentage (EP) = (Residual value*100)/Experimental shrinkage value (8.33)

Coefficient of variation (CV) = Standard deviation / Mean (8.34)

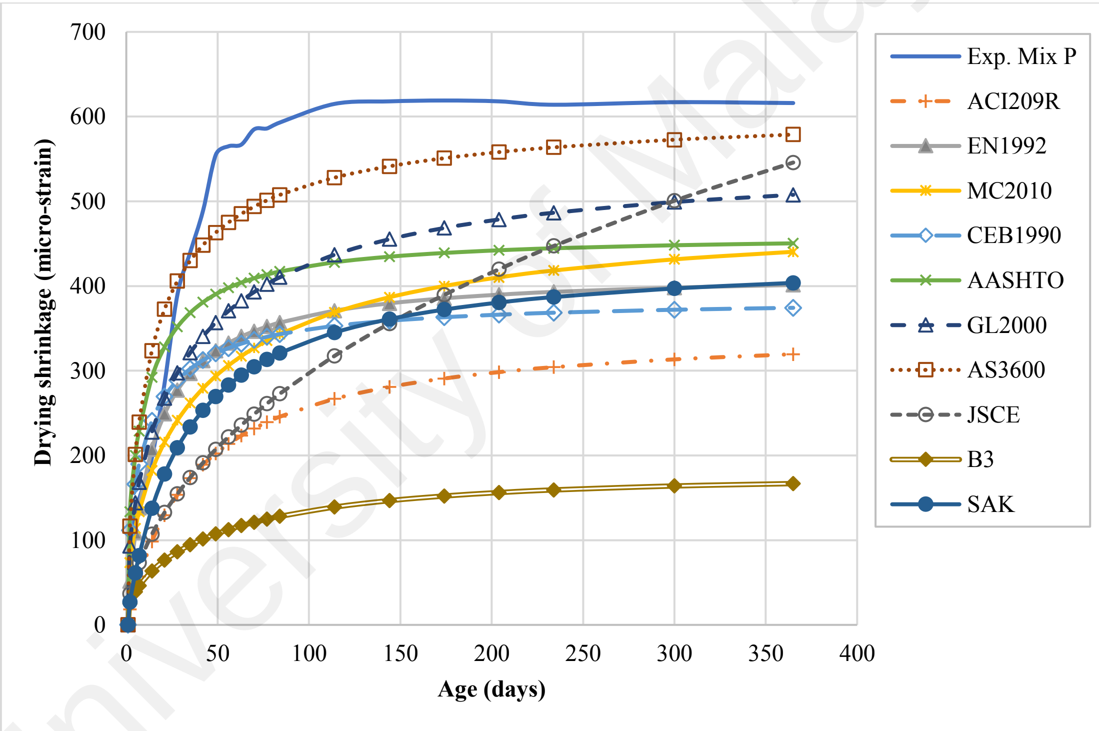


Figure 8.11: Comparison of drying shrinkage of OPS concrete (mix P) to the prediction models

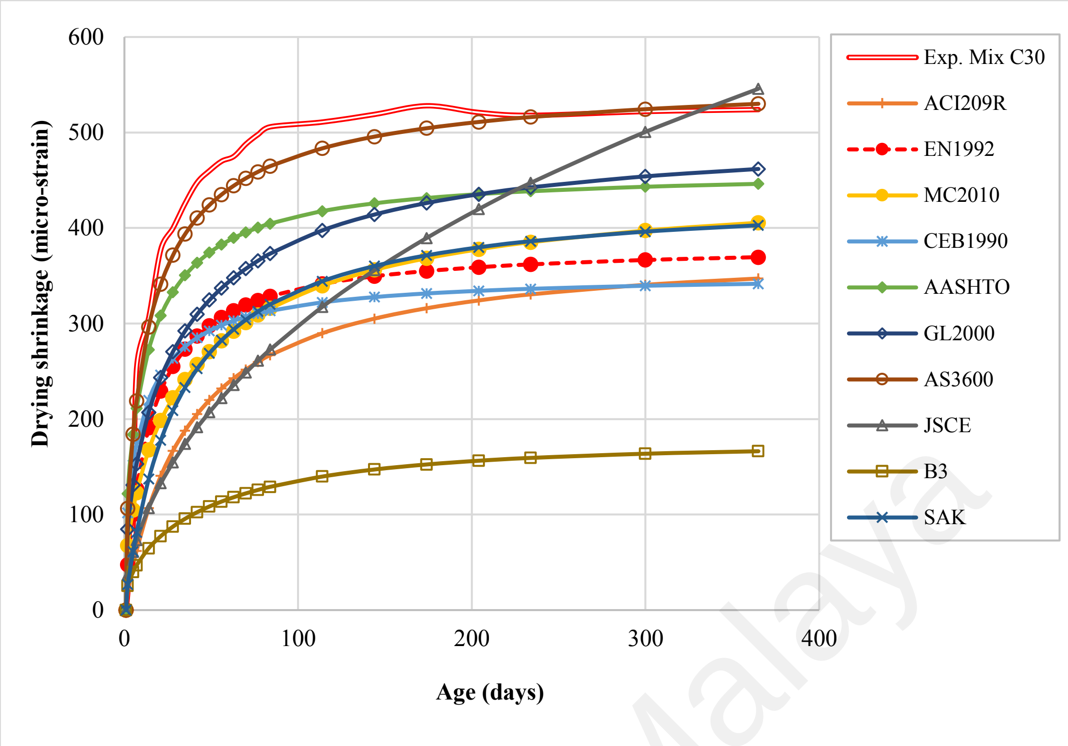


Figure 8.12: Comparison of drying shrinkage of OPS-OPBC concrete (mix C30) to the prediction models

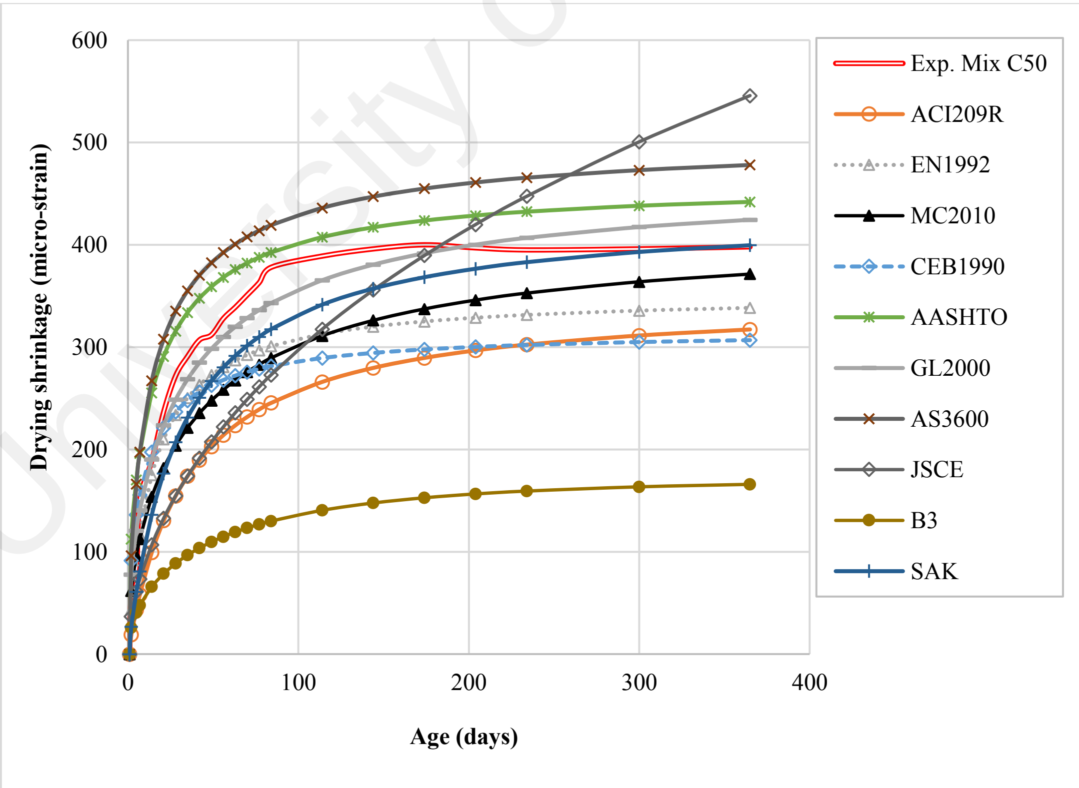


Figure 8.13: Comparison of drying shrinkage of OPS-OPBC concrete (mix C50) to the prediction models

The summary of the error percentage and the coefficient of variation for all the mixes in early ages are presented in Table 8.5 and Table 8.6, respectively. For OPS concrete (mix P), based on the EP and CV tests, the EN1992 and MC2010 models were the best predictors of the drying shrinkage, although the EN1992 model overestimated the results and the MC2010 model underestimated the shrinkages. The ascending rank of the remaining model predictions for the OPS concrete is ordered as GL2000, CEB/FIP, SAK, AASHTO-LRFD, AS3600, JSCE, ACI209R and B3. On the contrary, for OPS-OPBC concrete (mix C30), the models were selected based on the designed values of the EP and CV. The AS3600 was found as the best predictor for the concrete in early ages. However, the AASHTO model also predicted almost 90% similar results with the experimental shrinkage values, although these models underestimated the shrinkage values. The ascending rank of the remaining prediction models is ordered as CEB/FIP, GL2000, EN1992, MC2010, SAK, JSCE, ACI209R and B3. In addition, in the analysis of the best EP and CV, the prediction models for the OPS-OPBC concrete (C50) was observed. The GL2000 model was found as the best indicator for the concrete with an accuracy of approximately 100%, as shown in Table 8.6. However, the CEB/FIP and the EN1992 models predicted highly similar results with the experimental shrinkage values with an accuracy of approximately 94%, whereas the remaining models predicted the shrinkage results with the higher difference in the rank as MC2010, AASHTO-LRFD, SAK, AS3600, JSCE, ACI209R and B3. In early ages, for all the three mixes, the ACI209R and B3 models predicted lower results compared to the remaining models. This finding may be due to the type of concretes and the aggregates, as these models were majorly proposed for the normal weight concretes.

Table 8.5: Error percentage analyses for the mixes at early-ages (14 days)

Mixes	Prediction models									
	ACI209R	EN1992	MC2010	CEB/FIP	AASHTO	GL2000	AS3600	JSCE	SAK	B3
P	-50.25	5.07	-7.61	22.33	48.22	15.22	64.46	-45.68	-29.94	-67.51
C30	-65.03	-37.58	-45.09	-28.10	-11.11	-32.35	-3.26	-65.03	-55.22	-78.75
C50	-47.09	-7.40	-18.52	4.23	34.92	0.53	41.26	-43.38	-28.04	-65.07

Table 8.6: Coefficient of variation analyses for the mixes at early-ages (14 days)

Mixes	Prediction models									
	ACI209R	EN1992	MC2010	CEB/FIP	AASHTO	GL2000	AS3600	JSCE	SAK	B3
P	0.47	0.035	0.055	0.14	0.27	0.10	0.34	0.41	0.24	0.72
C30	0.68	0.32	0.41	0.23	0.08	0.27	0.023	0.68	0.53	0.91
C50	0.43	0.054	0.144	0.029	0.21	0.003	0.24	0.39	0.23	0.68

Table 8.7 and Table 8.8 show the summary of the error percentage and the coefficient of variation for the mixes in long-term ages (365 days). For mix P, based on the EP and CV results, the AS3600 and JSCE models are the best predictors with an accuracy approximately 90%-96%, although these models underestimated the shrinkage results. The same models (AS3600 and JSCE) were also found as the best predictors for the C30 concrete mixture with the accuracy of approximately 96%-100%. However, the GL2000, AASHTO and MC2010 models predicted the shrinkage results for mixes P and C30 with an accuracy in the range of approximately 70&-80%, although these models underestimated the predictions. The ascending rank of the remaining model predictions for the mixes P and C30 are ordered as SAK, EN1992, CEB/FIP, ACI209R and B3. For the C50 concrete mixture, four models can be considered the best predictor, where the SAK model predicted 100% similar results to the experimental values, whereas the GL2000, MC2010 and AASHTO models predicted similar results to the experimental

values with an accuracy of approximately 89%-94%. The SAK model is so simple to use. Moreover, more complex shrinkage models may not necessarily be more accurate than the simple models. The ascending rank of the remaining predicted models is ordered as EN1992, AS3600, ACI209R, CEB/FIP, JSCE and B3. The B3 model exhibited poor and non-conservative results of the shrinkage prediction for all the mixes, and this model is almost complex. Based on the results of this study, the accuracy of shrinkage prediction models depends on the type of aggregates and the compressive strengths used in the concrete mixes. Thus, the effects of types of aggregates such as OPS and OPBC should be considered in the shrinkage prediction models.

Table 8.7: Error percentage analyses for the mixes at long-term ages (365 days)

Mixes	Prediction models									
	ACI209R	EN1992	MC2010	CEB/FIP	AASHTO	GL2000	AS3600	JSCE	SAK	B3
P	-48.05	-34.90	-28.57	-39.28	-26.94	-17.53	-6.00	-11.36	-34.41	-72.88
C30	-33.78	-29.39	-22.52	-34.73	-14.88	-11.83	1.14	4.19	-23.09	-68.32
C50	-20.35	-14.82	-6.78	-22.86	11.05	6.53	20.10	37.18	0.50	-58.29

Table 8.8: Coefficient of variation analyses for the mixes at long-term ages (365 days)

Mixes	Prediction models									
	ACI209R	EN1992	MC2010	CEB/FIP	AASHTO	GL2000	AS3600	JSCE	SAK	B3
P	0.44	0.29	0.235	0.345	0.22	0.13	0.043	0.085	0.29	0.81
C30	0.28	0.24	0.18	0.30	0.11	0.088	0.008	0.029	0.18	0.73
C50	0.16	0.11	0.049	0.18	0.07	0.044	0.12	0.22	0.003	0.58

8.4 Conclusions

The development of the drying shrinkage strains for the OPS and OPS-OPBC lightweight concretes (mixes P, C30, and C50) were compared with the analytical results. Ten different prediction models (ACI209R, EN1992, MC2010, CEB/FIP1990, AASHTO-LRFD, GL2000, AS3600, JSCE, SAK and B3) were selected from the standards and researchers.

1. The error percentage (EP) and coefficient of variation (CV) equations were used to achieve the best prediction model for the lightweight concretes. In early ages, almost all the prediction models estimated similar results with the experimental curves. However, at later ages, only few models showed closer results. For OPSC, based on EP and CV values, the AS3600 and JSCE models were found as the best predictors with an accuracy of approximately 90%-96%, although these models underestimated the shrinkage results.
2. For OPS-OPBC50% concrete, four models can be considered the best predictors. The SAK model predicted 100% similar results to the experimental values, whereas the GL2000, MC2010 and AASHTO models predicted similar results with the experimental values with an accuracy of approximately 89%-94%. The SAK model is so simple to use. Moreover, more complex shrinkage models may not necessarily be more accurate than the simple models.
3. By contrast, the B3 model exhibited poor and non-conservative results of the drying shrinkage prediction for all mixes, and this model is more complex. Based on the results of this study, the accuracy of shrinkage prediction models depends on the type of aggregates and the compressive strengths of the concrete mixes. Thus, the effects of types of aggregates such as OPS and OPBC should be considered in the shrinkage prediction models.

CHAPTER 9: SHEAR BEHAVIOUR OF PALM OIL BY-PRODUCT LIGHTWEIGHT AGGREGATE CONCRETE BEAMS WITHOUT SHEAR REINFORCEMENT

9.1 Introduction

Shear behaviour is one of the most argumentized issue in the structural performance of the concrete members (Kong & Evan, 1998). Shear behaviour was first investigated about a century ago by (Morsch, 1909), who reported that a reliable definition of the shear behaviour of reinforced concrete (RC) beams without shear reinforcement is unavailable. However, shear behaviour can also be considered as brittle failure under a shear force, with the development of diagonal cracks in the span (Yang, 2014). According to classic beam theory, the maximum shear stress acts in the neutral axis of the beam section, which places the first principal stress in the diagonal direction and opens diagonal cracks. Therefore, such type of failure can be considered as shear failure. The use of reinforcement in concrete has several advantages, such as chemical and freeze-thaw resistance, lower time-dependent deformations, high modulus of elasticity etc. Apart from these advantages, concrete is more predisposed to shear failure, which mostly appears without any notification. A number of researchers have investigated the shear behaviour of reinforced concrete (RC) beams without shear reinforcement and reported several factors affecting the shear performance of RC members (Bukhari & Ahmad, 2008; Reineck & Kuchma, 2003). The main factors include strength of concrete, type of aggregate, shear-span to effect-depth ratio (a/d), loading type, longitudinal steel ratio (ρ) and end support conditions.

The lightweight concrete (LWC) is weak under shear, which is a main result of low tensile strength. The primarily failure in lightweight aggregate concrete (LWAC) occurs from the bond failure between the aggregate and the cement matrix and from the breaking of lightweight aggregate (LWA) (Alengaram, Jumaat, Mahmud, & Fayyadh, 2011).

According to Kong and Evans (1998), the shear strength of concrete is the most controversial topic in the structural performance of concrete members. Clarke (2002) studied the shear behaviour of RC beams in both with and without shear reinforcement. The concrete was made from two types of artificial LWAs, which have 28-day compressive strengths of 30 and 50 MPa and dry densities of 1700 and 2000 kg/m^3. The reduction factor of 0.9 was proposed for beams without shear reinforcement. These LWA concrete beams with the shear reinforcement can also carry about 90% capacity compared with the conventional concrete beams. Weigler and Karl (1980) utilized the lightweight expanded clay aggregates (LECA) and produced the lightweight aggregate foamed concrete (LWAFC) with 28-day compressive strengths of 12-30 MPa and the densities of 1200-1600 kg/m^3, successfully developing a structural grade LWAFC with thermal insulating characteristics. Later, Jumaat, Alengaram, and Mahmud (2009) investigated four oil palm shell foamed concrete (OPSFC) beams with the 28-day compressive strength and density of 20 MPa and about 1600 kg/m^3, respectively, where the OPSFC beams has higher shear capacity than conventional concrete beams and has more flexural and shear cracks in the absence of shear reinforcement.

Recently, Alengaram et al. (2011) investigated the shear performance of palm kernel shell concrete (PKSC) beams. The shear strength of grade 30 PKSC concrete with a density of 1850 kg/m^3 was about 24% higher than NWC, with the former establishing a better interlock of aggregates that produce shorter jagged cracks in comparison with the longer plain cracks of the latter. The bond breakdown in concrete, brought by the frequent failure of weaker OPS grains in PKSC arises from the concave and convex surfaces of the OPS (Alengaram et al., 2008b). Mahmud et al. (2009) also reported that mineral admixtures in the OPS concrete increases the mechanical properties and the bonding. Mohammed et al. (2013) investigated the shear behaviour of seven reinforced OPBC concrete beams, where the compressive strengths and air-dried densities of the selected concretes range at 20-40 MPa and about 1820-1840 kg/m^3, respectively. Upon consideration of shear-span to effective-depth ratio (a/d), the tensile reinforcement ratio (ρ), and compressive strength of the concretes the structural behaviour and the failure modes of the OPBC reinforced concrete beams were similar to normal weight RC beams.

The shear strength of LWC beams was not highly investigated compared to the conventional gravel concrete. Only few studies are available, where researchers investigated the shear performance of the oil palm shell (OPS) concrete. The oil-palm-boiler clinker (OPBC) aggregate is a new waste material from the oil palm industry, which still needs further structural investigations. Therefore, this research focuses on investigating the shear performance of OPS and OPS-OPBC reinforced concrete beams through the experimental work.

The main aim of this study is to use OPS and OPBC aggregates in producing different structural grades of LWC ranging from 25 to 45. The dried density below 2000 kg/m^3 was targeted and the shear behaviour of LWC without shear reinforcement was investigated. As reported by Aslam et al. (2016b), the OPS and OPBC can be used as an LWA in the concrete. The bulk density of the OPBC aggregate is about 40-45% lower than the NWA, but about 23-30% higher density than OPS and coconut shell. In addition, the smooth convex and concave surfaces of the OPS aggregate weaken the bond with the cement matrix (Mannan et al., 2002; Okpala, 1990; Shafigh et al., 2011). Aslam et al. (2016) studied the mechanical properties of green, environmentally-friendly, and high-strength structural LWAC blends produced by incorporating high volume solid waste materials (OPS and OPBC) from the palm oil industry. Though the literature on the mechanical properties of OPS and OPS-OPBC concretes is available, the shear behaviour of OPS-OPBC reinforced concrete beams is a neglected area.

9.2 Experimental Programme

9.2.1 Materials

Ordinary Portland cement (OPC) and Fly Ash (FA) were used as a binder in this investigation. Grade 48 OPC conforms to MS522, Part-1: 2003 and has a specific gravity of 3.14 and Blaine surface area of 3510 cm^2/g. The locally available FA of Class F according to ASTM: C618 has a specific gravity of 2.18 and Blaine surface area of 7290 cm^2/g. The chemical composition of the cement and fly ash is shown in Table 9.1.

Portable drinking water was used for mixing and curing of the concrete. According to BS-5075, Sika ViscoCrete with <0.1 total chloride ion content and cement compatibility was used as a super-plasticizer (SP) to control the workability of concrete. Local mining sand with the fineness modulus, specific gravity, and maximum grain size of 2.89, 2.68 and 4.75 mm, respectively, was used as a fine aggregate.

The OPS and OPBC were collected from the local palm oil industry and were used as a coarse aggregate. Both aggregates were stored at the open atmosphere for about 6-8 months for proper drying and removal of fibers from the OPS surface. OPS aggregates were washed in the concrete mixer to remove the oil and impurities from the surface. Whereas, the OPBC aggregates were crushed using a crushing machine. After that the aggregates were sieved to achieve the same grading. The physical properties of the selected aggregates are shown in Table 9.2.

Table 9.1: Chemical composition of fly ash (FA) and cement (OPC)

Oxide compositions	wt. (%)	
	OPC	FA
SiO_2	19.80	64.60
CaO	63.40	1.00
Fe_2O_3	3.10	4.00
MgO	2.50	0.66
AL_2O_3	5.10	20.9
SO_3	2.40	0.30
K_2O	1.00	1.20
Na_2O	0.19	0.32
LOI	1.80	5.10

Table 9.2: Physical properties of the aggregates

Properties	Coarse aggregates	
	OPS	OPBC
Specific gravity	1.19	1.69
Compacted density (kg/m^3)	610	860
24-h water absorption (%)	20.5	7.0

9.2.2 Mix Proportions

Details of the mixture proportions are shown in Table 9.3. Three sets of LWAC mixes were designed by using OPS and OPBC lightweight aggregates. Three grades of concrete were designed, in each grade, with all mixes containing the same aggregate volume, cement and SP contents. Each set contains three mixes, and set-1 was designed only by using OPS as coarse aggregate. Three concrete mixes of grades 25, 35, and 45 were designed in this set to investigate the OPS concrete strength on the shear capacity of the beams. For set-2 and set-3, the OPS aggregate in the set-1 mixes was replaced with 30% and 50% volumes of OPBC aggregate, respectively.

This study has three reasons for incorporating OPBC aggregate in OPS concrete. First, OPS concrete is very sensitive to the poor curing environment. However, OPBC contribution in OPS concrete significantly reduces the sensitivity of the latter to poor curing environment (Aslam et al., 2016). Second, OPS concrete has very high drying shrinkage strain, which seriously affects the structural performance of the beams.

Although, Aslam et al. (2016a) investigated the drying shrinkage behaviour of the OPS-OPBC concretes, the OPBC contribution in OPS concrete with slightly reduced water-to-cement ratio significantly reduces the drying shrinkage strain of OPS concretes. Third, incorporating OPBC in OPS concrete reduces the smooth surface of the aggregates in the concrete. Given the convex and concave nature of OPS aggregate on both faces, the bond between the aggregate (shells) and the cement matrix is not strong enough to sustain higher loads. Therefore, partial substitution of OPS and OPBC illustrated the blended effect of LWAs on the shear capacity of RC beams without shear reinforcement.

All OPS and OPS-OPBC concrete mixes were prepared in the rotating drum mixer. Initially, the materials were weighed under dry conditions. The OPS and OPBC lightweight aggregates were pre-soaked in water for 24 hours and were dried in the laboratory environment to obtain the saturated surface dry condition. For each mix proportion, OPC, FA and aggregates were mixed for 2 minutes, before adding 70% water with SP for another 3-minute mixing time. The rest of the water was added and the final mixture was rotated for another 5 minutes. The properties of the fresh concretes were measured (Table 9.3).

Table 9.3: Concrete mix proportions

Sets	Mix code	Cement (kg/m^3)	FA (kg/m^3)	Water (kg/m^3)	SP (kg/m^3)	Sand (kg/m^3)	Coarse aggregate (kg/m^3)		Slump (mm)
							OPS	OPBC	
Set-1	P1	432	48	202.10	2.25	774	851	-	150
	P2	450	50	181.34	5.00	779	856	-	195
	P3	468	52	154.00	4.79	785	863	-	200
Set-2	C30-1	432	48	202.10	2.25	774	596	255	190
	C30-2	450	50	181.34	3.80	779	599	257	180
	C30-3	468	52	161.75	4.79	785	604	259	200
Set-3	C50-1	432	48	202.10	2.25	774	425.5	425.5	190
	C50-2	450	50	181.34	3.80	779	428	428	215
	C50-3	468	52	161.75	4.79	785	431.5	431.5	185

9.2.3 Preparation of Specimens

A total of nine reinforced 1800 mm × 125 mm × 250 mm concrete beams were designed and prepared by using OPS and OPS-OPBC concretes. A clear cover of about 25 mm was provided for all beams. The typical arrangement of reinforcement in the beams is shown in Figure 9.1. All beams were designed with the same tensile reinforcement ratio without shear reinforcement. While shear reinforcement was only provided at the beam supports and the loading points to clearly observe the load impact on the shear span of the beams, the main aim is to observe the shear capacity of different concrete grades and the effect of different types of LWAs on shear behaviour. The steel reinforcements with the same specifications were collected from one source for all the beams. A high-yield strength of f_y = 545 N/mm^2 deformed bar was used for flexural reinforcement, whereas the mild steel reinforcement with the yield strength of f_{yv} = 300 N/mm^2 was used for shear reinforcement. The shear-span to effective-depth ratio (a/d) of 2.87 was used for all beams without shear reinforcement.

All beams and the concrete specimens were cast in steel moulds. Compressive strength and density of concrete were observed from 100 mm cube specimens. These specimens were cast in parallel with the beams and cured under standard curing, however, the beams were cured with water sprinkling and gunny bags curing method. The properties of the concretes and the reinforcement details used in the beams are provided in Table 9.4.

Table 9.4: Properties of the specimens / beams

Sets	Concretes	Beam designation	Oven-dry density (kg/m^3)	28-day compressive strength (MPa)	Tensile reinforcement ratio
Set-1	OPSC	P1	1702	26.70	0.0128
	OPSC	P2	1757	36.60	0.0128
	OPSC	P3	1791	45.95	0.0128
Set-2	OPS-OPBCC30%	C30-1	1795	29.94	0.0128
	OPS-OPBCC30%	C30-2	1824	37.34	0.0128
	OPS-OPBCC30%	C30-3	1903	46.96	0.0128
Set-3	OPS-OPBCC50%	C50-1	1853	29.79	0.0128
	OPS-OPBCC50%	C50-2	1941	39.05	0.0128
	OPS-OPBCC50%	C50-3	1966	47.66	0.0128

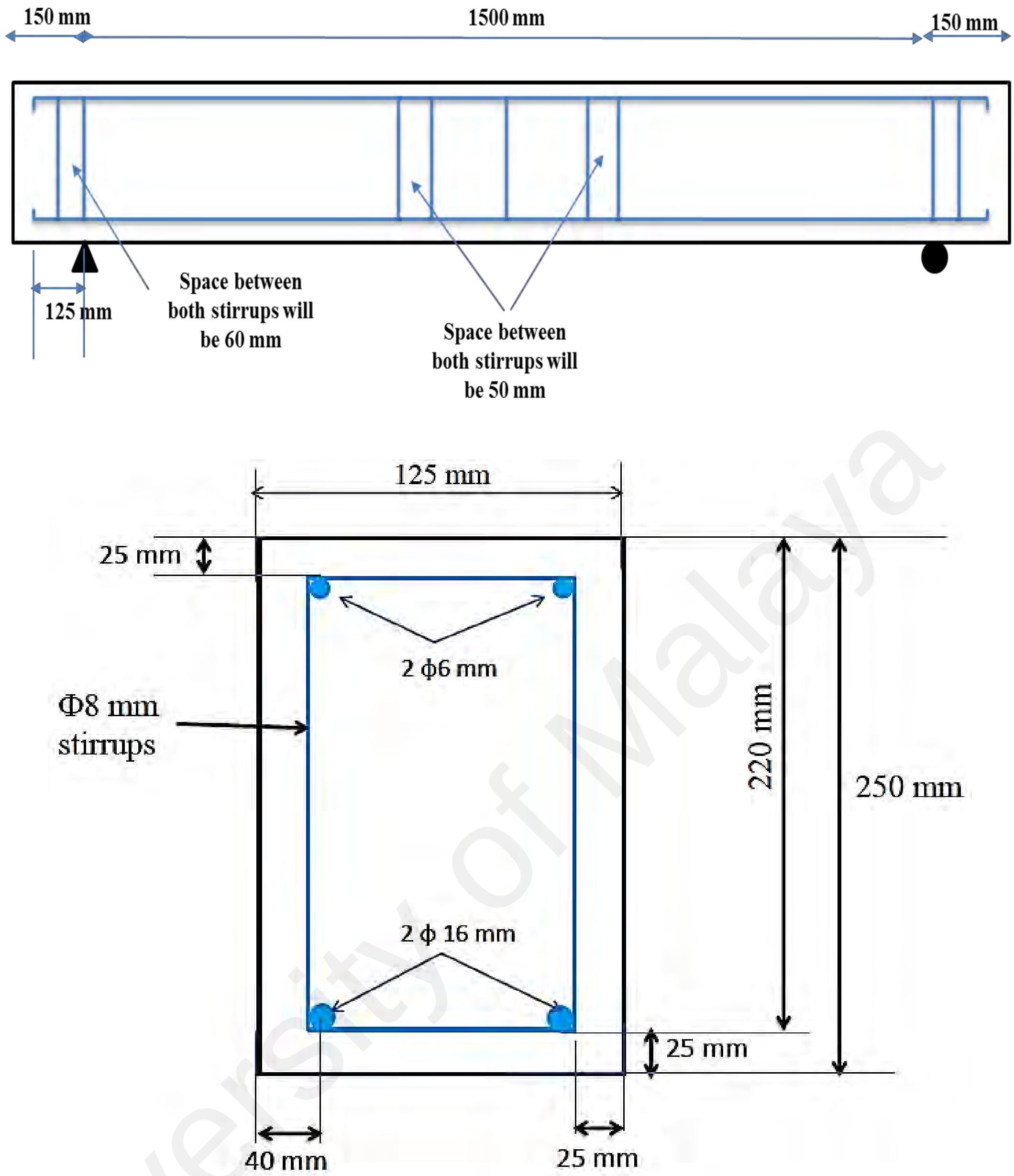


Figure 9.1: Reinforcement details of the beams without shear reinforcement

9.2.4 Instrumentation and Testing

All concrete beams were tested under four-point loading with a constant effective span. Details of the experimental setup are illustrated in Figure 9.2. The Instron Universal Testing Machine (IUTM) with a maximum cell capacity of 600 kN was used to test all the beams. The mid-span deflection of the beams was measured by using linear voltage differential transducers (LVDT). The beams were tested under 12 intervals with actuator rates of 5 kN/min and 0.01 mm/sec during load control and the displacement control, respectively. In addition, both the shear and the flexure crack widths were measured by using a Dino-Lite digital microscope with an accuracy of 0.02 mm at each load increment.

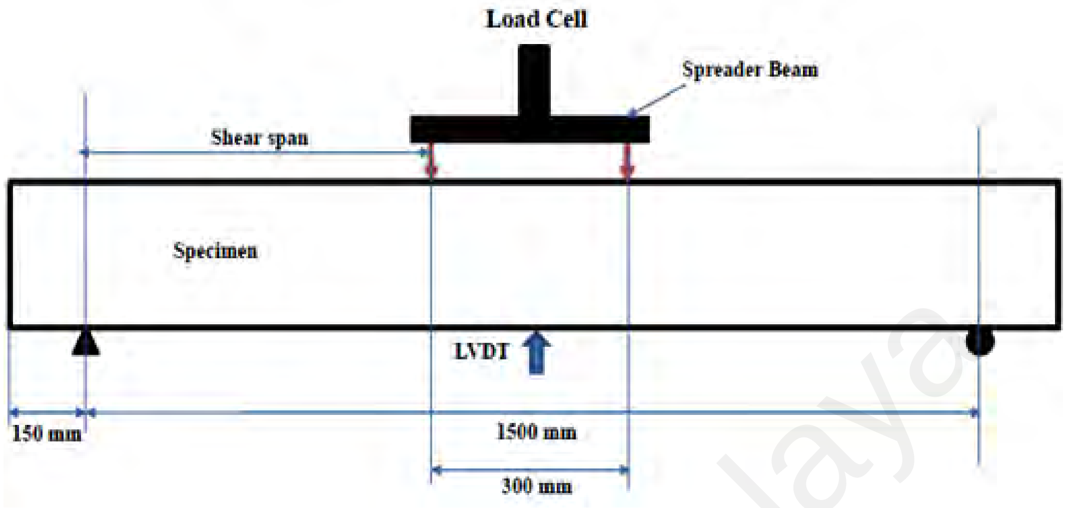


Figure 9.2: Test setup and Instrumentation

9.3 Results and Discussions

9.3.1 Structural Behaviour

9.3.1.1 OPS concrete beams with different grades

Set-1 contains three reinforced OPS lightweight concrete beams as P1, P2 and P3 with different strength grades of 25, 35, and 45, respectively. The structural behaviour with the respective deflection of the set-1 LWC beams are shown in Figure 9.3, where higher concrete strength translates to the higher load carrying capacity. P1 beam has a maximum load capacity of about 68 kN and a maximum deflection of about 8.075 mm, whereas P2 and P3 beams have an ultimate capacity of about 86 and 113 kN at deflections of about 9.75 and 16.27 mm, respectively. All beams showed similar behaviour from early loading up to 20 kN. However, OPSC (P2 and P3) beams showed better results at heavier loads. On average, P2 and P3 beams have ultimate strengths of about 21% and 39% higher than the P1 beam, respectively.

Wight and MacGregor (2009) reported that the flexure capacity is generally considered first in the design of RC members, leading to the member size and reinforcement to provide necessary moment resistance. The limitations are only allowable during flexural reinforcement to ensure the gradual failure of the structure. However, the shear is the only failure in concrete, that is frequently sudden and brittle. Therefore, the shear strength must be higher than flexural strength in the shear design of RC members. As seen in Figure 9.3, P1 and P2 beams showed very gradual shear failure in comparison with the P3 beam.

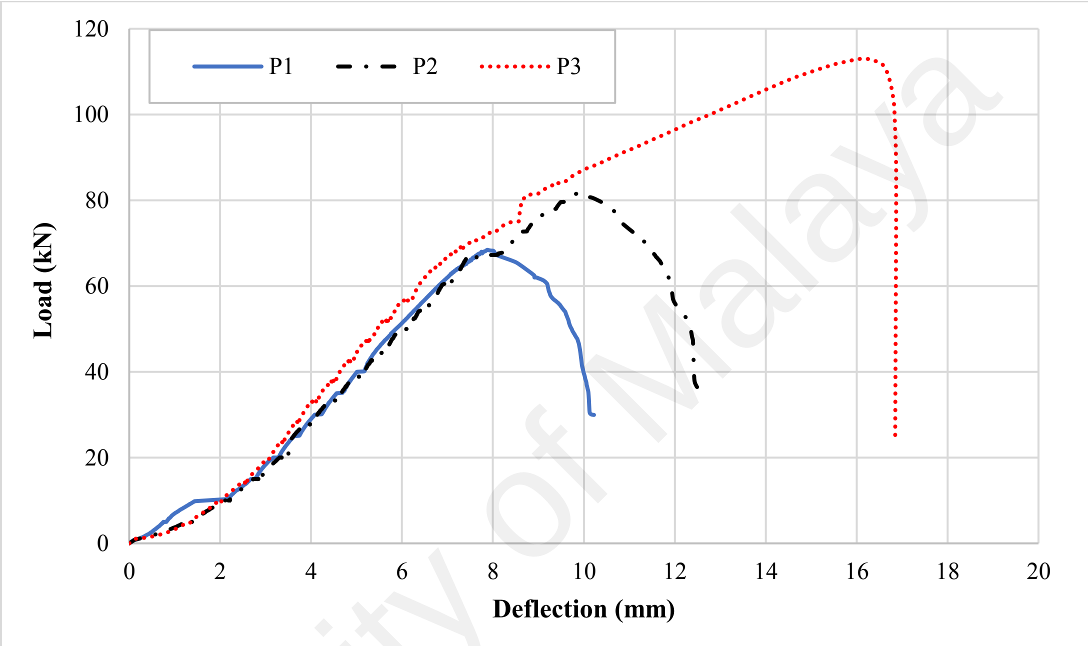


Figure 9.3: Structural behaviour of OPS concrete beams (set-1) without shear reinforcement

9.3.1.2 OPS-OPBC concrete beams with different grades

Similar to set-1 of OPSC beams, set-2 and set-3 were designed with the same strength grades by using OPS and OPBC aggregates. Each set in set-2 and set-3 concrete mixes contain three beams with the modification of replacing OPS aggregates in set-1 mixes with 30% and 50% OPBC aggregates, respectively. Therefore, the concrete mixes can be considered as OPS-OPBC30% and OPS-OPBC50% concrete beams. The behaviour of set-2 and set-3 reinforced LWC beams are shown in Figure 9.4 and Figure 9.5. As shown in Figure 9.4, all set-2 mixes showed similar behaviour up to 60 kN loads, however, the higher-grade concrete beams sustained higher loads under higher deflection as the loading increases. The beam C30-1 showed the ultimate capacity of about 73 kN at a deflection of 10.7 mm, whereas, the load carrying capacity of the beams also increases as the grade of the same concrete increases beyond 25 MPa. The beams C30-2 and C30-3 showed the highest ultimate loads of about 85 and 116 kN at deflections of 12.3 mm and 16.8 mm, respectively. The ultimate load of beam C30-3 was about 37% and 27% higher than C30-1 and C30-2 beams, respectively.

As seen in Figure 9.5, all set-3 mixes showed similar behaviour up to 75 kN load with an average difference of about 4%. However, the ultimate capacity of the beams with higher grade strength also increases as the loading increases. The C50-1 beam showed the ultimate capacity of about 77 kN with a deflection of about 12.2 mm, whereas, the load carrying capacity of the beams also increases as the grade of the same concrete increases beyond 25 MPa. C50-2 and C50-3 beams showed the highest ultimate loads of about 97 and 121 kN at the deflections of 15.5 mm and 16.97 mm, respectively. The ultimate load of C50-3 was about 36% and 20% higher than C50-1 and C50-2 beams, respectively.

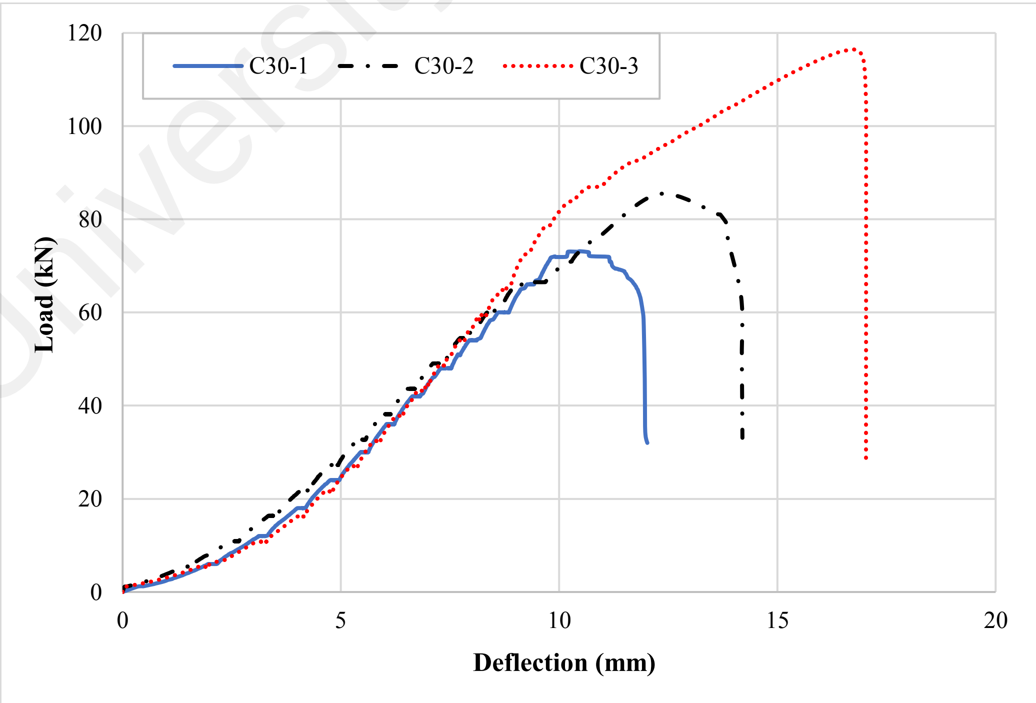


Figure 9.4: Structural behaviour of OPS-OPBC30% concrete beams (set-2) without shear reinforcement

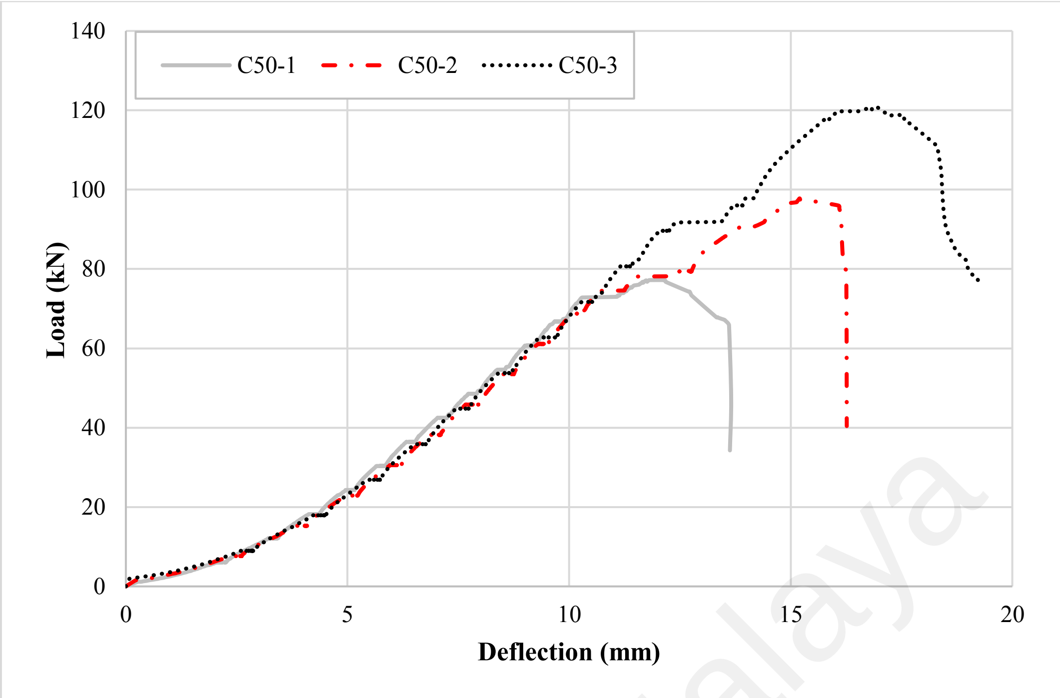


Figure 9.5: Structural behaviour of OPS-OPBC50% concrete beams (set-3) without shear reinforcement

9.3.1.3 Comparison of structural behaviour between OPSC and OPS-OPBC concrete beams

Figure 9.6 shows a comparison of the structural behaviour of OPSC and OPS-OPBCC beams, though the difference between the OPSC and OPS-OPBCC beams is not significant. OPSC beams showed lower ultimate capacity compared with OPS-OPBC concrete beams. OPS concrete beams (P1 and P2) were ductile after failure load, however the failure load of P3 beam was more brittle. Similarly, the C30-1 and C30-2 beams also showed the ductile behaviour, whereas the C30-3 suddenly failed after the ultimate loading capacity. The structural behaviour of the set-3 beams is a bit different from set-1 and set-2 specimens. The beams C50-1 and C50-2 showed a bit ductile-natured behaviour after the ultimate loading capacity, however, C50-3 beam showed total ductile behaviour as its strength grade increases.

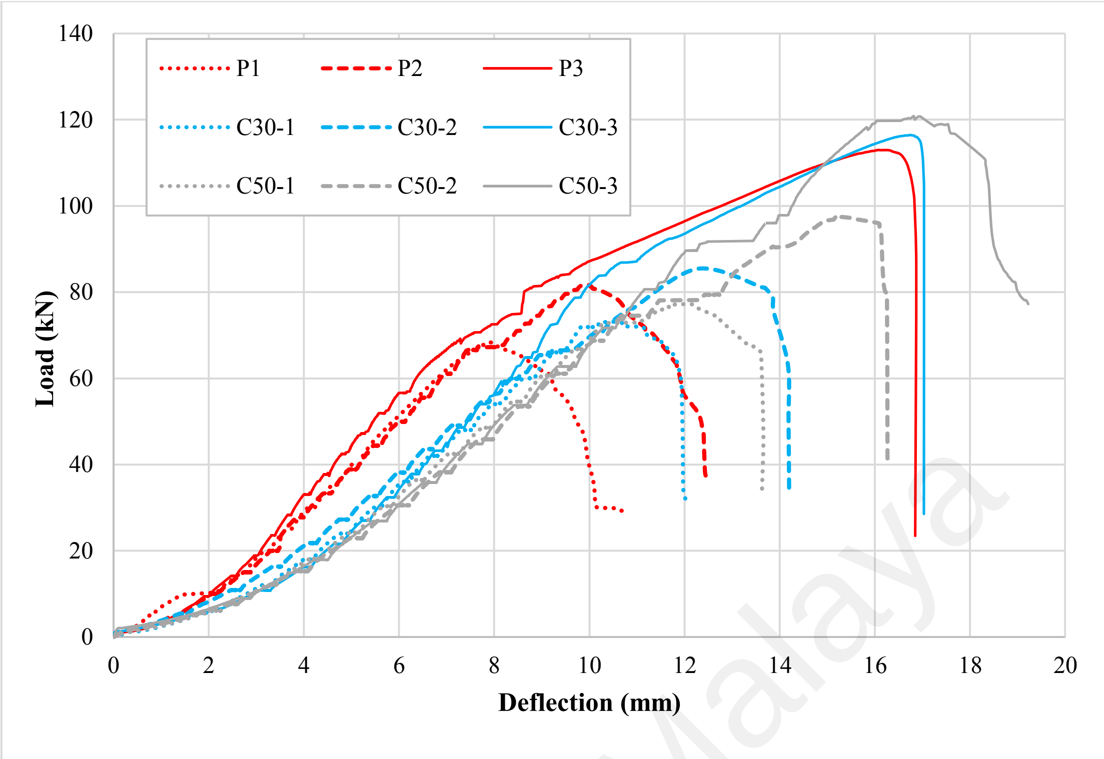


Figure 9.6: Structural behaviour of all the beams without shear reinforcement

The main reasons behind better structural performance of OPS-OPBC concrete beams are mainly from the types of aggregates. As OPS aggregate is smooth by virtue of its concave and convex faces, which weakens the bond between the aggregate and the cement matrix (Aslam et al., 2016; Mannan et al., 2002; Okpala, 1990). Therefore, the failure of the concrete results from the debonding of OPS aggregates from the cement matrix, and from the inability of the shells fully utilize their potential strength. As reported by Alengaram et al. (2011), the primary failure in most PKS lightweight concrete arises from the breaking of both the aggregate and the bond between the aggregate and cement matrix. The bond breakdown in concrete brought by the frequent failure of the weak OPS grains arises from the concave and convex surfaces of the OPS (Alengaram et al., 2008b). Mahmud et al. (2009) also reported that mineral admixtures in OPS concrete increases the mechanical properties and the bonding. Recently, Aslam et al. (2016) demonstrated the contribution of OPBC to the significant improvement of compressive strength in OPS concrete under standard curing condition. Therefore, incorporation of the OPBC aggregate in OPS concrete has a significant effect on the bond behaviour between aggregates and the cement matrix. Based from the structural behaviour, the OPBC in the OPS concrete increases the shear resistance of the beams. However, 50% OPBC in OPS with higher strength grade concrete showed better ductile behaviour among all beams, which mainly arose from the use of blended aggregates in the beam. Jumaat et al. (2009) recommended that a good aggregate interlock in the concrete leads to a higher shear resistance in the OPSC because the shapes of the various particles range from angular to parabolic.

The ultimate load carrying capacity of all the beams with the respective deflections is shown in Figure 9.7. Incorporation of 50% OPBC in OPS concrete increases the structural performance of the OPSC beams. As seen from the figure, concrete grade increases along with the increase of deflection and the gradual increase in the angles (α, β and ɵ) of the deflection lines. The set-1 concrete beams showed the lowest angle (α), whereas, the set-3 concrete showed highest angle (ɵ) between the deflection line.

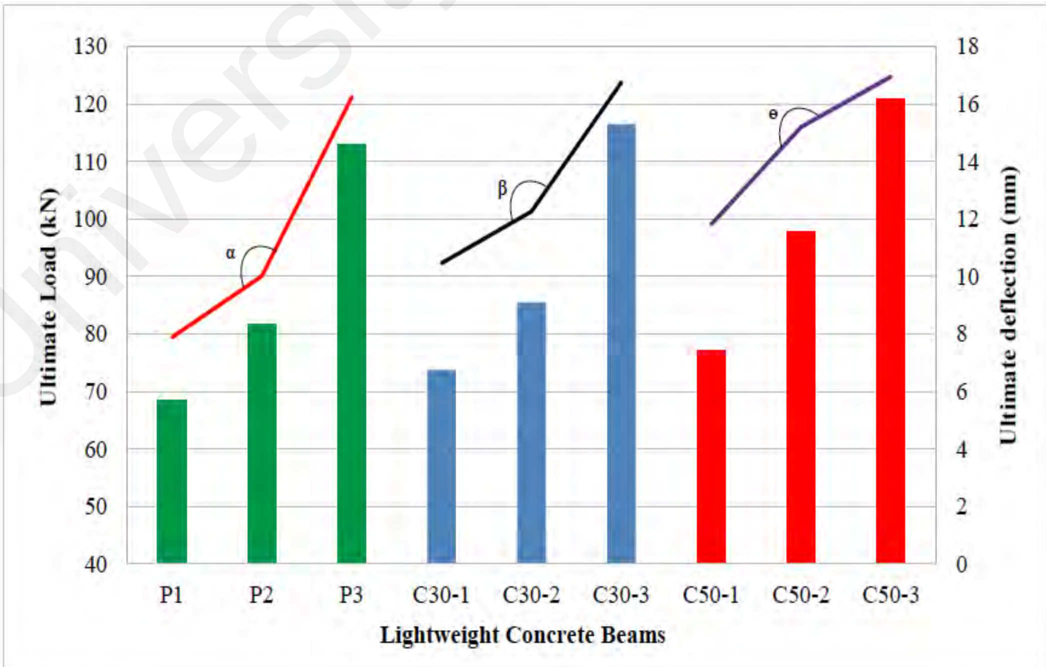


Figure 9.7: Ultimate load capacity of all the beams with the respective deflections

9.3.2 Shear Behaviour of Beams

Table 9.5 summarizes the first flexure crack load, first shear cracking load, ultimate shear capacity, number of cracks at failure, failure modes, and the ratio of ultimate shear capacity to the density of concrete. The incorporation of the OPBC aggregate significantly increased the ultimate shear capacity of the OPS beams. Among grade 25 concrete beams, the ultimate shear capacity of the C50-1 beam is about 7 and 11% higher than C30-1 and P1 beams, respectively. Among grade 45 concrete beams, C50-3 beam showed the highest ultimate shear capacity at 60.4 kN, which, on the average, was about 37% and 24% higher than G25 and G35 OPS-OPBC50% concrete beams, respectively.

The total number of cracks in each beam was also measured at the time of failure. As seen in Table 9.5, all OPS concrete beams showed a higher number of cracks than the OPS-OPBC beams because of two major possibilities. First, the number of cracks at the time of failure reduces as the concrete grade improves. Second, OPBC in OPS concrete reduces the total number of cracks in the beams. The C50-3 beam incorporated with 50% OPBC and highest concrete grade of 45 showed the lowest number of total cracks at the time of failure. Therefore, the incorporation of OPBC aggregate in the OPS concrete significantly improved the bonding between the aggregate and the cement matrix which resulted in the lower number of cracks.

The ratio of ultimate shear strength to the oven-dry density of the concrete beams is also presented in Table 9.5. Beams with the same grade strength showed almost similar shear strength to density ratios. Although, the performance of the OPS-OPBC concretes improved from the incorporation of OPBC in the OPS concrete, the strength to density ratio was found almost similar in each grade. In comparison with the reported ratio of 19.7 $N/mm^2/kg/m^3$ in NWC beams with the ultimate shear strength of 46 kN (Alengaram et al., 2011), the strength to density ratio of grade 45 beams in this present study is higher by about 36%.

Table 9.5: Summary of Experimental results

Beam designation	First flexural crack load (kN)	First shear crack load (kN)	Ultimate shear capacity (kN)	Number of cracks at failure	Failure modes	Ultimate shear / Concrete density (Nm^3 / kg mm^2) x 1000
P1	10	12	34.2	29	Shear	20.0
P2	15	18	40.8	28	Shear	23.2
P3	10	24	56.5	26	Shear	31.5
C30-1	10	23	36.9	23	Shear	20.6
C30-2	20	28	42.7	22	Shear	23.4
C30-3	14	30	58.3	21	Shear	30.6
C50-1	12	23	38.6	20	Shear	20.8
C50-2	20	29	48.9	18	Shear	25.2
C50-3	15	35	60.4	13	Shear	30.7

9.3.3 Cracking Behaviour

The early flexural cracks in all beams may appear as a result of lower mechanical properties such as splitting tensile and modulus of rupture of the concretes. However, the incorporation of OPBC in OPS concrete increases the splitting tensile strength and the modulus of rupture of the latter (Aslam et al., 2016). The diagonal tension cracks occurred in all the beams, which appeared closer to the support and propagated towards the loading point. The cracking patterns between the supports and the loading point of all the nine beams tested under the shear are given in Figure 9.8, Figure 9.9 and Figure 9.10. All beams displayed initial flexural cracking in the flexural zone. The OPSC (set-1) and OPS-OPBC30% (set-2) beams had a greater number of both flexural and shear cracks and closer spacing than the OPS-OPBC50% (set-3) concrete beams. As seen in Table 9.5, the set-1 and set-2 beams under shear showed almost similar numbers of flexural and shear cracks, whereas, the number of similar (flexural and shear) cracks is significantly lower in set-3, which suggests that OPBC in the OPS concrete significantly increases the bonding between the tension reinforcement and the OPS-OPBC concrete. Alengaram et al. (2011) and Jumaat et al. (2009) reported that the PKSC and OPSFC beams with and without shear reinforcement had a higher number of the flexural and shear cracks and closer spacing in comparison with the corresponding NWC beams. Although, Mohammed et al. (2013) later did not report the cracking behaviour of palm-oil-clinker (POC) RC beams the study specified that the reinforced POC beam showed similar failure mode to that of the NWC beam.

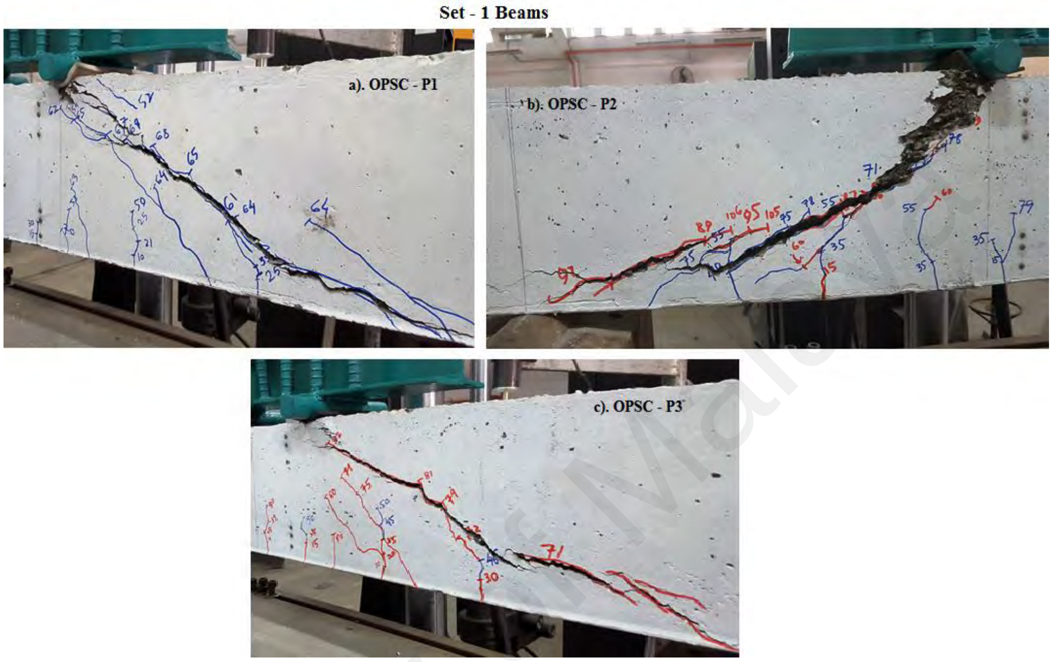


Figure 9.8: Cracks pattern and Failure modes of the set-1 (OPS) lightweight weight concrete beams

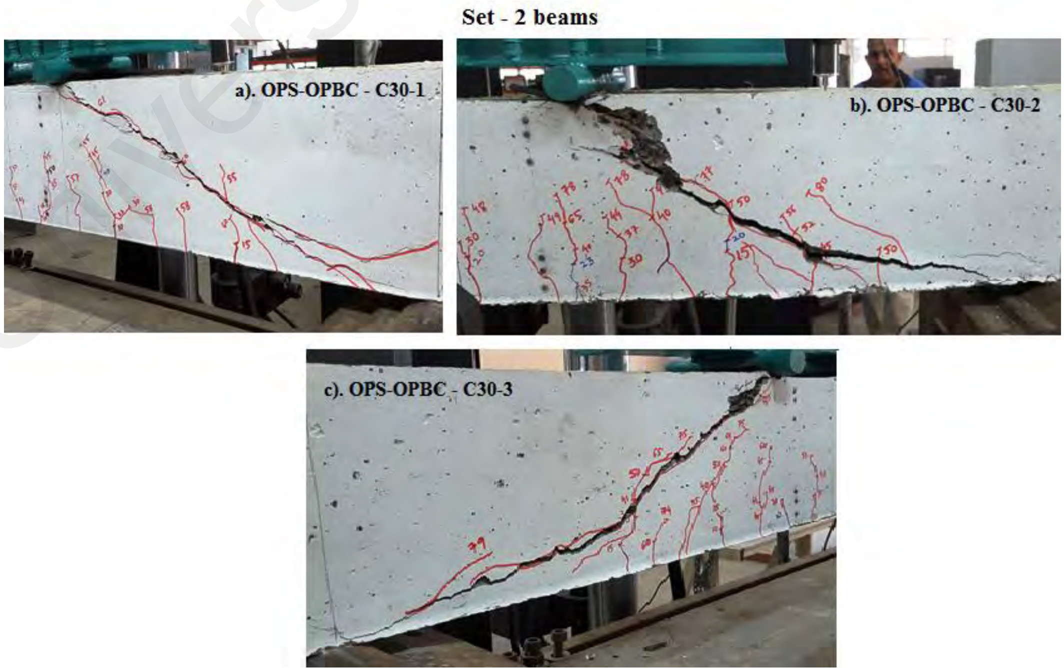


Figure 9.9: Cracks pattern and Failure modes of the set-2 (OPS-OPBC30%) LWC beams

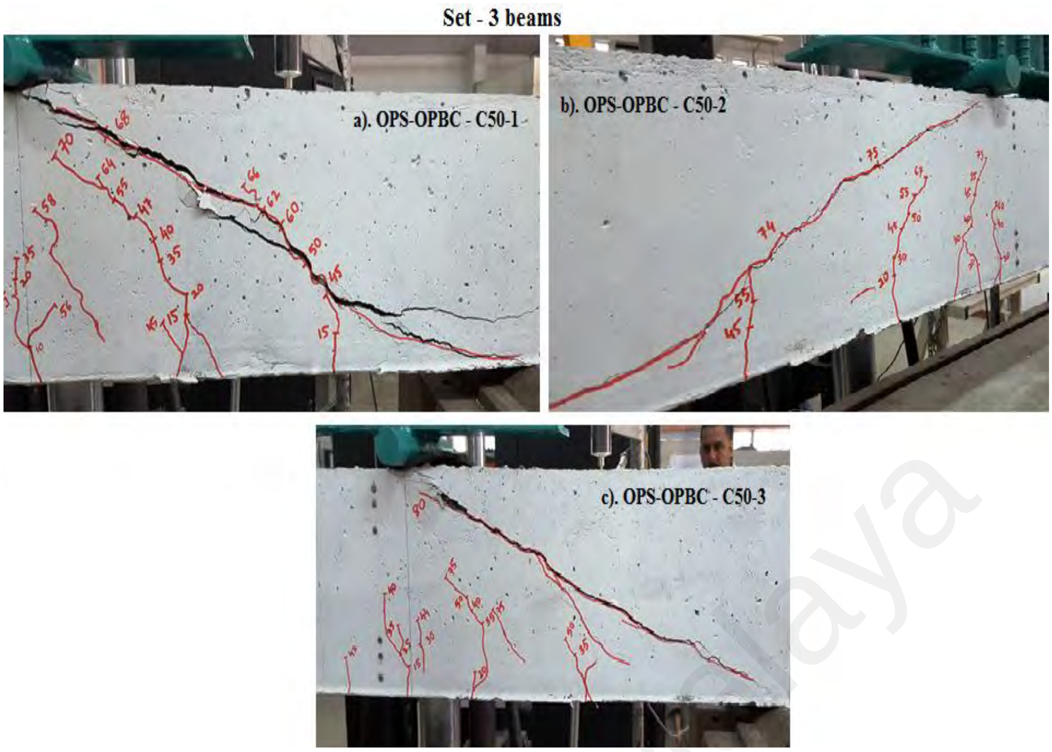


Figure 9.10: Cracks pattern and Failure modes of the set-3 (OPS-OPBC50%) LWC beams

Under service loading conditions, the crack width of the first crack appearance in all beams was continuously observed at every interval up to 60 kN, and the final crack widths were measured exactly after failure with the development of crack widths shown in Figures 9.11 to 9.13. Based from the LWC beams with grade 25 strength presented in Figure 9.11, all beams showed similar behaviour. However, P1 beam showed higher crack widths in the range of 0.065-0.749 mm, whereas, C30-1 and C50-1 beams showed crack width ranges of about 0.025-0.672 mm and 0.022-0.597 mm, respectively. At the failure stage, the crack width of the P1 beam was about 11% and 20% higher than C30-1 and C50-1 beams, respectively. This observation was attributed to the incorporation of the OPBC aggregate in improving the performance of OPS-OPBCC beams in comparison with OPSC. The similar behaviour was also found for grade 35 concrete beams (Figure 9.12), where the maximum ranges of crack widths in P2, C30-2, and C50-2 beams were about 0.026-0.669 mm, 0.038-0.520 mm, and 0.038-0.477 mm, respectively. At the failure stage, the crack width of the P2 beam was about 22% and 29% higher than the C30-2 and C50-2 beams.

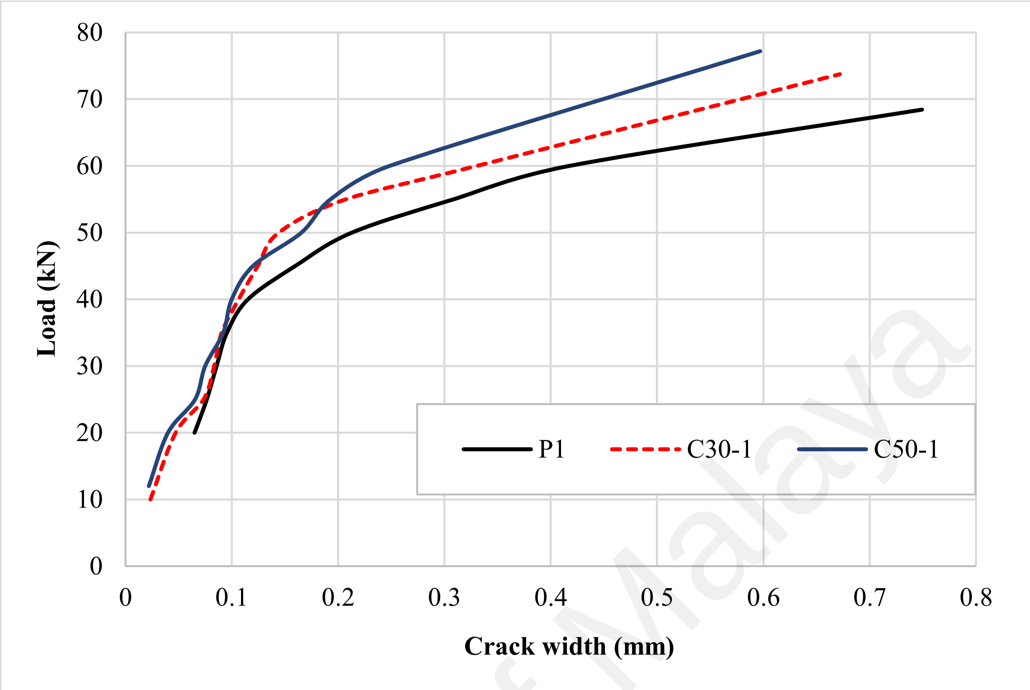


Figure 9.11: Development of cracks widths of Grade 25 concrete beams

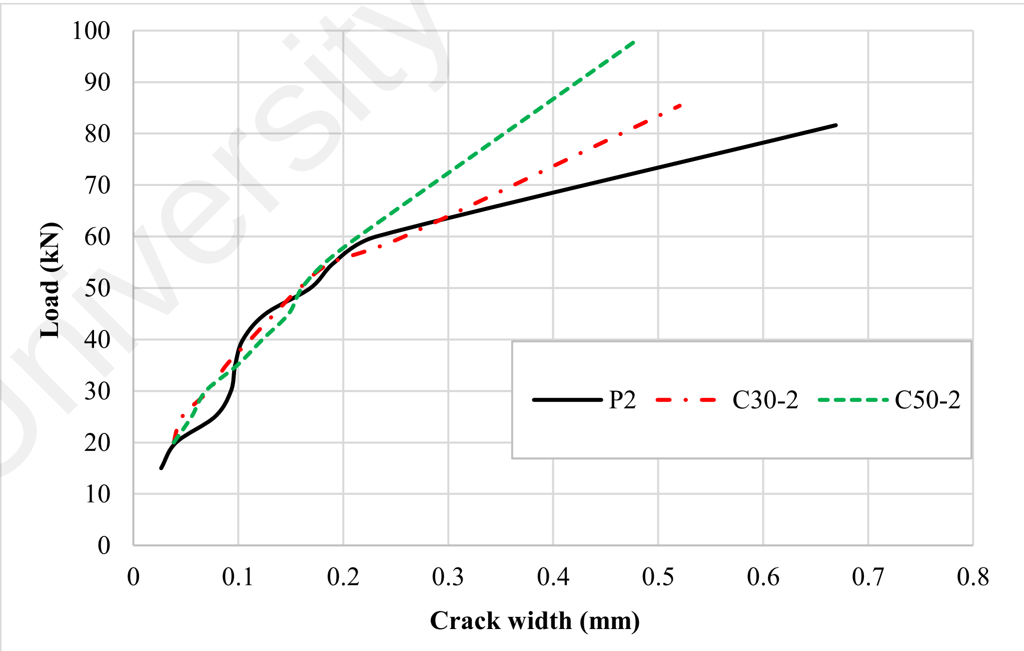


Figure 9.12: Development of cracks widths of Grade 35 concrete beams

Figure 9.13 can be described as the development of crack widths of the LWC beams with the grade 45 strength. At the failure stage, the P3, C30-3, and C50-3 beams showed the maximum crack widths at the range of 0.032-0.468 mm, 0.020-0.411 mm, and 0.018-0.389 mm, respectively. As the grade of the concrete increases, the maximum crack widths reduce. Comparatively, the crack width of the P3 beam was about 38% and 30% higher than the P1 and P2 beams. The crack width of C30-3 beam was about 39% and 21% higher than the C30-1 and C30-2. The crack width of the C50-3 beam was about 35% and 18% lower than C50-1 and C50-2 beams.

Generally, the crack widths in the RC members can be considered based on certain reasons such as leakage, appearance, and the reinforcement corrosion. ACI224.1R (2007) reported that clean smooth surfaces and cracks wider than 0.254 mm to 0.330 mm can easily lead to a public concern, despite the absence of any universal rule for the maximum crack widths. Prior to 1999, the ACI Code recommended 0.330 mm for the exterior exposure and 0.406 mm for the interior exposure as limits for the maximum crack widths (Gergely & Lutz, 1973), whereas the Euro-International Concrete Committee (CEB-Manual, 1985; CEB/FIP, 1993) limits the mean crack width to about 60% of the maximum crack width for the exposure conditions, duration of the loading condition, and the sensitivity of reinforcement to corrosion. In this study, a maximum of 60 kN service loading condition in grades 25, 35, and 45 concrete beams showed the crack widths in the range of 0.200 mm to 0.330 mm, which are almost equivalent to the maximum limits of cracks width suggested by ACI standards. However, the crack widths of the beams at failure stage were found higher than the suggested range.

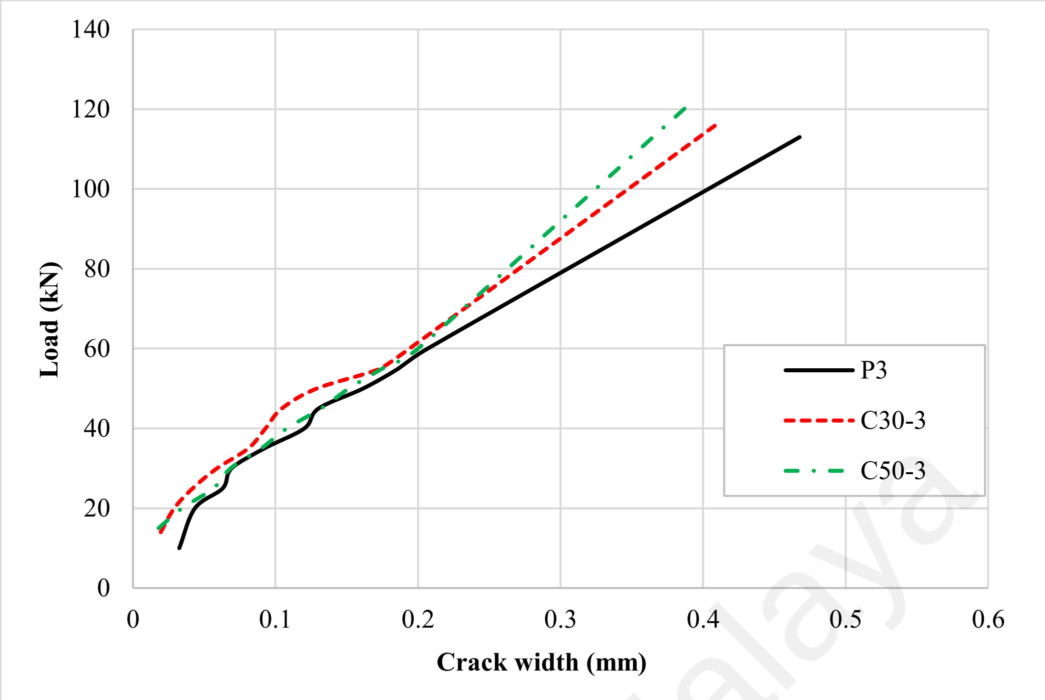


Figure 9.13: Development of cracks widths of Grade 45 concrete beams

9.3.4 Failure Modes

Failure modes of all the designed beams without shear reinforcement were shown in Figure 9.8, Figure 9.9, and Figure 9.10. In the flexural region, the initial flexural cracks appeared at about 16% of the ultimate strength. Table 9.5 summarizes the details of all beams such as first flexural crack load, first shear crack load, and failure modes. As seen in the figures, all beams showed pure shear failure without any anchorage failure. All beams were designed with the same a/d ratio of 2.87, therefore, the formation of diagonal tension cracks is the evident in these beams. Kong and Evans (1998) stated that the formation of diagonal cracks is independent in RC beams with a/d's of 1-2.5 instead of flexural crack development. In this investigation, all beams showed shear cracks independent of the flexural cracks. Alengaram et al. (2011) examined the shear behaviour of PKSC and NWC concrete beams with and without shear reinforcement, where PKSC beams without shear reinforcement failed in shear, whereas, some beams from both concretes showed additional anchorage failure, as a result of higher stress concentration near the supports.

Wight and MacGregor (2009) reported that short beam spans with a/d of 1-2.5 generally develop inclined cracks and carry additional loading after redistribution of internal forces, in part by an arch action. Bond failure along the reinforcement or crushing of compression zone, later referred to as the shear compression failure will cause the final failure of such beams. With the inclined crack generally spreading deeper into the beam rather than the flexural crack, failure occurs at less than the flexural moment capacity. Furthermore, the inclined cracks in beams with a/d ratio of 2.5-6 disrupt the equilibrium of the beams, which result in an inclined cracking load brought by beam failure. Beams with a/d ratio higher than 6, normally failed in flexure prior to the appearance of inclined cracks. However, all beams studied in this study showed a general shear failure.

9.3.5 Deflection

Figure 9.3 shows the mid-span deflection or the load-deflection behaviour of all the beams without shear reinforcement. The initial linear portion of the curvatures shows that the stiffness of the beams remains constant before the appearance of the flexural cracking. The performance of the OPSC beams is comparable with that of OPS-OPBC concrete beams. All lower to medium (25-35) strength concrete beams showed a consistent increment in the deflection with respective loading, whereas the higher strength (grade 45) OPS and OPS-OPBC30% concretes showed a sudden failure without any increment in the deflection. However, the failure of the same grade OPS-OPBC50% concrete was a ductile failure with the constant increase in the deflection. Comparatively, the deflections can be observed in two ways. First, if the concrete grade is considered, then improving the concrete grade also increases the deflections. Second, the contribution of OPBC aggregate in OPS concrete resulted in higher deflections and improved ductility of the OPS-OPBC beam with higher grade concrete. BS8110 limits the total deflection to span/250. However, the deflections of all beams at service loads were within the allowable limit of 9.6 mm.

In the load-deflection behaviour, the beam is initially un-cracked owing to its higher stiffness. However, the stiffness and the moment of inertia of the beams decreases as the cracking starts. Further cracking causes the yielding of steel reinforcement, leading to higher deflections with the little change of load Wight and MacGregor (2009). Hence, all beams in the early stage showed an elastic behaviour and the progressive reduction of flexural stiffness owing to the increase in crack widths and loads caused non-linear load-deflection.

9.3.6 Comparison of Experimental and Predicted Shear Strengths

The shear resistance of all the beams (V_c) was calculated using the equations proposed by ACI318 (2002) Eurocode 2 (EN2, 2004), Canadian Standard (CSA-A23.3, 1994), and Bazant and Kim (1984). According to ACI318 (2002), the shear resistance of non-prestressed reinforced concrete members without shear reinforcement can be calculated by using Eq. (9.1). However, this equation predicts the shear load in US-units. Therefore, the shear strength was converted into the SI-units, as seen in Eq. (9.2).

$$V_c = 2 * \lambda * \sqrt{f_c'} * b_w * d \quad \text{(US-Units)} \quad (9.1)$$

$$V_c = \left(\frac{\lambda * \sqrt{f_c'}}{6}\right) * b_w * d \quad \text{(SI-Units)} \quad (9.2)$$

where; V_c is the shear force, $\lambda = 1$, for NWC and lesser for LWCs.

The standard Eurocode 2 (EN2, 2004) normally considered the Variable Strut Inclination Method to estimate the shear strength of the concretes, generally allowing the designers to determine the economic amount of the shear reinforcement in the concrete members (Mosley, Hulse, & Bungey, 2012). The empirical expression provided by EC2 in calculating the shear resistance of the concrete ($V_{Rd,c}$) is mentioned in Eq. (9.3). This standard generally considers the 28-day cylinder compressive strength of the concrete, which can be converted from the 28-day cube compressive strength. This code also recommended that the partial safety factor (γ_c) of 1.50 is not considered in the calculation of shear strength. In addition, the EN2 recommended two different factors of ($C_{Rd,c}$) for the conventional and LWC. Introduction of the factor (η_1), which considers the concrete density (ρ), allows the proper estimation of the shear capacity of the beams. However, this factor may influence the shear resistance of the LWC, rather than the conventional NWC.

$$V_c = \left[C_{Rd,c} * k_c * \eta_1 * (100 * \rho_1 * f_{ck})^{1/3}\right] b * d \qquad (9.3)$$

where,

$C_{Rd,c} = \frac{0.18}{\gamma_c}$ for normal weight concrete (NWC); and

$C_{Rd,c} = \frac{0.15}{\gamma_c}$ for lightweight concrete (LWC);

$k_c = 1 + \sqrt{\frac{200}{d}} \leq 2.0$ where d is the depth in (mm);

$\eta_1 = 0.4 + \frac{0.6\rho}{2200} \leq 1.0$ where ρ is the concrete density in (kg/m^3);

$\rho_1 = \frac{A_s}{b_w d} \leq 0.02$

The Canadian standard (CSA-A23.3, 1994) proposed the shear strength prediction Eq. (9.4). The Canadian standard does not consider the effects of a/d and the tension reinforcement on the shear strength of concrete. According to the equation, the f_c' is the 28-day compressive strength of the concrete in MPa, whereas b_w and "d" are the effective width and depth of the effective cross-section in millimeter, respectively.

$$V_c = 0.2 * \sqrt{f_c'} * b_w * d \tag{9.4}$$

Bazant and Kim (1984) formulated the shear strength prediction model for the concrete members shown in Eq. (9.5). The model looks complicated because the equation considers several parameters in predicting the shear strength of concrete members. Aside from the diagonal shear failure of RC beams and the slabs without shear reinforcement, the model considers the size effect, which theoretically resulted from the dimensional analysis of the energy release rate during fracture propagation that disperses cracking zones at their front.

$$V_c = \left[0.54 * \sqrt[3]{\rho} * \left(\sqrt{f_c'} + 249\sqrt{\frac{\rho}{\left({}^{a}/_{d}\right)^3}}\right) * \left(\frac{1+\sqrt{{}^{5.08}/_{d_0}}}{\sqrt{1+{}^{d}/_{(25d_0)}}}\right)\right] b_w * d \tag{9.5}$$

where, f_c' is the 28-day compressive strength of the concrete; $b_w d$ is the width and depth of effective cross-section in mm; *a/d* is the shear span to the effective depth ratio, and ρ is the longitudinal reinforcement ratio.

The experimental and the predicted results of the shear strength of all beams is shown in Table 9.6. All prediction models selected from different standards estimated close results in the actual shear strength of the beams. The ratios of the experimental to the ACI318 model results for set-1, set-2, and set-3 beams are 1.41-1.78, 1.44-1.81, and 1.51-1.86, respectively. This ratios in EC 2 ranges at 1.38-1.85, 1.39-1.83, and 1.44-1.86 for set-1, set-2, and set-3 beams, respectively. The ACI318 and the EC2 models predicted very conservative results of ultimate shear strengths at an average ratio of 1.60. However, the model proposed by Canadian Standards predicted better results compared with ACI318 and EC2. The ratios of the experimental to the Canadian model results for the set-1, set-2, and set-3 beams were in the ranges of 1.18-1.48, 1.20-1.51, and 1.26-1.56, respectively. The ACI318 and EC2 models predicted conservative results in comparison with Canadian standards because the shear behaviour of the beams fails under brittle failure without any warning. The ultimate shear strength model proposed by Bazant and Kim (1984) predicted lower results in comparison with the experimental values. The ratios of experimental to the Bazant model results for the set-1, set-2, and set-3 beams range at 0.73-1.06, 0.77-1.10, and 0.81-1.13, respectively. Although, this model is covering several parameters in the prediction of ultimate shear strengths, however, this model is not considered for design purposes given its lower predictions. Alengaram et al. (2011) investigated the shear behaviour of PKSC beams, and reported the ratio between the experimental and the predicted shear strengths by BS8110, ACI318 and EC2 at the range of 1.57-2.83. All three standards underestimated the actual ultimate shear strength of the concrete beams with and without shear reinforcement. In this study, the Bazant and Kim (1984) model predicted the average ultimate shear strength at 8.5%, 32%, 42.7%, and 43% of the experimental values, Canadian model, EC2 model, and ACI model, respectively. As the concrete grade increases, the ratios of experimental to the predicted values of ultimate shear strength also increase in all the three sets.

Table 9.6: Comparison of experimental and predicted results for beams without shear reinforcements

Beams	Exp. Shear capacity (kN)	Predicted shear force (kN)				V_{Exp} / V_{ACI}	V_{Exp} / $V_{EC\ 2}$	V_{Exp} / V_{CAN}	V_{Exp} / V_{Bazan}
		ACI 318	EC 2	Canadian	Bazant				
P1	34.2	24.2	24.8	29.1	46.5	1.41	1.38	1.18	0.73
P2	40.8	28.4	28.0	34.0	50.0	1.44	1.46	1.20	0.82
P3	56.5	31.8	30.5	38.2	52.9	1.78	1.85	1.48	1.06
C30-1	36.9	25.6	26.4	30.1	47.7	1.44	1.39	1.20	0.77
C30-2	42.7	28.6	28.8	34.4	50.3	1.50	1.48	1.24	0.84
C30-3	58.3	32.1	31.8	38.5	53.3	1.81	1.83	1.51	1.10
C50-1	38.6	25.6	26.9	30.7	47.7	1.51	1.44	1.26	0.81
C50-2	48.9	29.3	30.2	35.2	50.8	1.67	1.62	1.39	0.96
C50-3	60.4	32.4	32.5	38.8	53.5	1.86	1.86	1.56	1.13
Average (%)						1.60	1.59	1.34	0.91

9.4 Conclusions

Three sets of the lightweight aggregate concrete (LWAC) mixes were designed using coarse waste materials from the palm oil industry, namely, oil palm shell (OPS) and oil-palm-boiler clinker (OPBC) to investigate the shear behaviour. Each set contains three concrete mixes of grades 25, 35, and 45. Set-1 was designed by only using OPS as a coarse aggregate, whereas in set-2 and set-3, replaced the OPS aggregate in the set-1 mixes with 30% and 50% volume of OPBC aggregate, respectively. Based on this investigation, several conclusions can be drawn:

1. In set-1 (OPSC beams), a stronger concrete indicated a higher load carrying capacity. The ultimate strengths of Grade 35 (P2) and grade 45 (P3) beams were about 21% and 39% higher than the grade 25 (P1) beam, respectively.
2. In set-2 (OPS-OPBCC30% beams), the ultimate load of beam C30-3 (grade 45) was the highest at about 37% and 27% higher than C30-1 (grade 25) and C30-2 (grade 35) beams, respectively.
3. In set-3 (OPS-OPBCC50% beams), all beams showed similar behaviour up to 75 kN. The ultimate load of beam C50-3 (grade 45) was the highest at about 36% and 20% higher than the C50-1 (grade 25) and C50-2 (grade 35) beams, respectively.
4. Comparatively, the OPSC beams have lower ultimate capacity in comparison with OPS-OPBC concrete beams. The OPSC beams up to grade 35 showed a ductile behaviour after ultimate load carrying capacity. However, the beam showed brittle failure without any warning as the grade of OPSC increased. On the other hand, incorporation of 50% OPBC aggregate in the OPSC significantly improves the structural performance and the failure nature of the beam.
5. All OPSC beams showed a higher number of flexural and shear cracks than OPS-OPBC beams for two main reasons. First, the concrete grade improves as the

number of cracks at the time failure reduces. Second, the OPBC in OPSCs reduces the total number of cracks in the beams.

6. The ratio of ultimate shear strength to the oven-dry density of the concrete beams was also observed. Beams with the same grade strength showed almost similar shear strength to the density ratios. This ratio was also compared with the ratio of NWC beam selected from literature, where the strength to density ratio of the grade 45 concrete beams in this study is 36% higher than NWC beams.
7. Under 60 kN service loading conditions, grades 25, 35, and 45 concrete beams showed the crack widths of 0.20-0.3305 mm which is almost equivalent to the maximum cracks width limit suggested by ACI standards.
8. The beams in this study were designed with the same a/d of 2.87 and without the shear reinforcement. Therefore, all beams showed pure shear failure without any anchorage failure.
9. The experimental results of ultimate shear strength of the beams were also compared with the predicted results. All prediction models selected from different standards estimated close results with the actual shear strength of the beams. On average, the ACI 318 and the EC 2 models predicted conservative results of the ultimate shear strengths at an average ratio of 1.60.

CHAPTER 10: CONCLUSIONS AND RECOMMENDATIONS

10.1 Conclusions

The possibility of using oil-palm-boiler clinker (OPBC) and oil palm shell (OPS) as lightweight aggregate in the concrete to produce green, sustainable and high strength blended coarse lightweight aggregate concrete. The following are the general conclusions based on the cumulative conclusions given in the papers:

10.1.1 Workability

i. It is possible to produce green, sustainable and high strength lightweight aggregate concrete with high workability without any segregation.

ii. By replacing oil palm shell by oil-palm-boiler clinker in a oil palm shell concrete workability increases as the percentage of OPS substitution increases.

iii. Substitution of cement by fly ash up to 10% also increases the workability of the OPS and OPS-OPBC lightweight concretes.

10.1.2 Density

i. OPBC grain is about 42% heavier than an OPS grain, inclusion of OPBC in OPS concrete increased the density of concrete by about 2-4%. However, even at the 50% substitution level, the density of OPS-OPBC concrete was still in the acceptable range for structural lightweight aggregate concretes.

ii. The density of concrete is found to decrease with the substitution of the OPBC with OPS aggregates. The substitution between 20 to 40 % could turn high-strength semi-lightweight OPBC concrete into high-strength lightweight aggregate concrete.

iii. The lowest 28-day oven dry density was measured for artificial lightweight aggregate concrete (leca concrete) was about 1590 kg/m^3, which was significantly lower than the OPS, OPS-OPBC, OPBC and normal weight concretes.

10.1.3 Compressive Strength

i. It is possible to produce high strength lightweight aggregate concrete by using waste materials as blended coarse aggregates.

ii. The contribution of oil-palm-boiler clinker (OPBC) in OPS concrete significantly improved the compressive strength under standard curing. At the substitution between 20-40% levels, grade 35 OPS concrete was transferred to grade 40.

iii. The substitution of 50% OPS with OPBC in OPS concrete the compressive strength was significantly improved. By this substitution, grade 35 concrete with the oven dry density of about 1800 kg/m^3 was transferred to grade 50 concrete with the oven dry density of about 1950 kg/m^3.

iv. In continuous moist curing, the set (1) concrete mixes containing OPBC from 20 to 50%, the compressive strength of grade 30 OPS concrete could be transformed to grade 40. However, the set (2) concrete mixes with 30-50% OPBC aggregate had significant improvement in the compressive strength and grade 50 OPS lightweight concrete was achieved.

v. The 28-day compressive strength of the artificial lightweight aggregate concrete (leca concrete) was found about 25 MPa, which is significantly lower than the OPS and OPS-OPBC lightweight concretes.

10.1.4 Effect of Curing Regimes on Compressive Strength

i. Incorporating oil-palm-boiler clinker (OPBC) aggregate into OPS concrete reduces the sensitivity of compressive strength to lack of curing. OPS concrete (without OPBC aggregate) under air-drying showed a reduction of about 25% compared to continuous water curing while OPS concrete containing 30-50% OPBC only showed a reduction of about 5%. This may be due to the better performance of OPBC lightweight aggregates for internal curing.

ii. Partial early curing improves compressive strength more than air-drying because it is more effective when OPBC content is higher. However, if partial early curing occurs though watering a few times a day (like 2T2D and 2T6D), it is recommended that this method is not used continuously over several days due to the negative effect of the wetting-drying condition on concrete.

iii. The concretes containing OPBC and OPS lightweight aggregates showed lower compressive strength under air drying. However, six days moist curing after demoulding is enough to achieve equivalent compressive strength under full water curing.

10.1.5 Splitting Tensile Strength

i. The 28-day splitting tensile strength of concretes containing OPBC from 10-50% was found in the range of 3.05-3.31 MPa, which is in the usual range for structural lightweight aggregate concrete.

ii. The substitution of 50% OPS with OPBC in OPS concrete significantly improved the splitting tensile strength of the concrete.

iii. Although, OPBC concretes containing OPS had lower tensile strengths compared to OPBC concrete, the splitting tensile strength was found to be in the usual range for structural lightweight aggregate concrete.

10.1.6 Flexural Strength

i. The 28-day flexural strength of OPS-OPBC concretes ranged from 4.48 to 5.38 MPa, which was 12-15% of the 28-day compressive strength. These ratios are equivalent to the normal weight concrete ratio.

ii. The substitution of 50% OPS with OPBC in OPS concrete the flexural strengths significantly improved. By this substitution, grade 35 concrete with the oven dry density of about 1800 kg/m^3 was transferred to grade 50 concrete with the oven dry density of about 1950 kg/m^3.

iii. Although, OPBC concretes containing OPS had lower flexural strengths compared to OPBC concrete, moreover, the OPBC-OPS concretes showed flexural strength and flexural/compressive strength ratio similar to NWC and high strength lightweight aggregate concretes.

10.1.7 Specific Strength

i. The incorporation of OPBC aggregate in OPS concrete increased the specific strength of the concrete. The specific strength for a NWC with a dry density of 2350 kg/m^3 and 28-day compressive strength similar to mix OPS-OPBC50% is 21915 N.M/kg, which is about 17.5% lower than the specific strength of OPS-OPBC concrete. Therefore, it can be said that OPS-OPBC concretes even shows better performance compared to NWC of the same grade.

10.1.8 Modulus of Elasticity

i. The modulus of elasticity of OPS concretes increased about 18-24% with the incorporation of OPBC aggregates. OPS-OPBC concretes containing more than 20% OPBC aggregate had a modulus of elasticity in the normal range for structural lightweight aggregate concretes.

ii. The modulus of elasticity of grade 35 OPS concrete is very low compared to normal concrete and structural lightweight aggregate concrete at the same compressive strength. However, the incorporation of OPBC in OPS concrete significantly enhanced this property. The modulus of elasticity of OPS-OPBC concrete is in the normal range of structural concretes.

iii. The modulus of elasticity of the OPBC concrete was reduced by the incorporation of the OPS aggregates. The reduction of the modulus of elasticity at high volume substitution of 60% was found to be very significant.

10.1.9 Ductility Performance

i. The contribution of OPS in OPBC concrete up to 60% could significantly increase the strain at the peak stress (ε_0) of the concrete mixes. This shows that OPS lightweight aggregate can improve ductility performance of lightweight aggregate concretes.

10.1.10 Water Absorption

i. The water absorption of OPS concrete was reduced by increasing the percentage of OPBC aggregates. All OPS-OPBC mixes showed initial and final water absorption of less than 3% and 10%, respectively, and can be considered good quality concretes.

ii. Water absorption of the OPBC concrete increased linearly by increasing OPS content in this concrete. This occurs due to higher water absorption of OPS compared to OPBC. The results of the current study suggest that the substitution level should be limited to 20-40% (by volume).

10.1.11 Drying Shrinkage

i. The drying shrinkage of OPS and OPS-OPBC concretes is similar at early ages. However, OPS-OPBC concrete showed significantly lower drying shrinkage compared to OPS concrete after one month.

ii. The contribution of OPBC in OPS concrete showed similar shrinkage results of the drying shrinkage to the control OPS concrete at early ages. However, the long-term shrinkage was significantly reduced compared to control OPS concrete.

iii. The contribution of OPBC in OPS concrete along with small reduction on water to cement ratio significantly reduced the drying shrinkage compared to control OPS concrete. The mix C-50(2) showed the lowest shrinkage value among all the mixes and it was about 40% lower than the OPS concrete.

iv. The incorporation of OPBC aggregate in OPS concrete shows better results under 7-day moist curing compared to structural lightweight aggregate concretes made of lightweight aggregates such as lytag, expanded shale and sintered fly ash.

v. The OPS-OPBC concrete mixes showed lower early-age drying shrinkage under 7-day curing compared to uncured specimens. The results also confirm that the curing time is essential to minimize the shrinkage at early ages. Therefore, the pre-saturation of aggregates reduces the early-age shrinkage and delays the drying shrinkage.

vi. In early ages, all the prepared lightweight concretes (OPSC, OPBCC and LECA concrete) showed closer results of drying shrinkage to the conventional concrete (NWC). Later at 56 days age, the OPSC showed significantly higher drying shrinkage than the other concretes. Whereas, the OPBCC and LECA concrete showed similar trend to the conventional concrete.

vii. Long-term (after one year of drying), the OPS concrete on average showed 40% higher drying shrinkage strains compared to the other concretes.

viii. Compared to OPSC, the aggregates in other concrete mixes (C, L and N) showed better bonding with the cement paste in Interfacial Transaction Zone (ITZ) and reduced the formation of micro-cracks, which resulted significantly lower drying shrinkage strains.

ix. OPSC showed significantly higher drying shrinkage results under 7-day moist curing, which is about 47%, 41% and 39% higher than the shrinkages of normal weight, LECA and OPBC concretes, respectively. Whereas, the average difference between the shrinkage values of NWC and OPBCC and LECA concretes was found in the range of 10-13%.

x. The OPS, OPBC and conventional concrete shows lower early-age shrinkage under 7-days curing, and the reduction in OPSC was higher than OPBCC and NWC. This also confirm that the curing time is essential to minimize the shrinkage at early ages. Whereas, in long-term shrinkage, the cured specimens showed higher drying shrinkage than the uncured ones, and the average difference for OPSC and LECA concretes was found almost similar of about 6%.

xi. The ACI drying shrinkage prediction model estimated lower results for the OPS concrete with the average difference of about 50%. However, for other concretes (mixes N, C and L), the ratio between the experimental and the predicted results was found in the range of 20-30%.

xii. The error percentage (EP) and the coefficient of variation (CV) equations were used to achieve the best prediction model for the lightweight concretes. It was observed that in early ages, almost all the prediction models estimated closer results to the experimental curves. However, at later ages only few models showed closer results. For OPSC, based on EP and CV values, the AS3600 and JSCE models were the found as best predictors with the accuracy of about 90-96%, although these models underestimated the shrinkage results.

xiii. For OPS-OPBC50% concrete, four models can be considered as the best predictor, in that the SAK model predicted 100% similar results to the experimental values, whereas, the GL2000, MC2010 and AASHTO models predicted closer results to the experimental values with the accuracy range of about 89-94%. The SAK model is so simple to use and it can be shown that more complex shrinkage models may not necessarily be more accurate than the simple models.

xiv. Whereas, The B3 model exhibited poor and non-conservative results of the drying shrinkage prediction for all the mixes, although, this model is more complex. Based on the results of this study, the accuracy of shrinkage prediction models depends on the type of aggregates and the compressive strengths of the concrete mixes. Thus, the effects of types of aggregates such as OPS and OPBC should be consider in the shrinkage prediction models.

10.1.12 Structural Performance

i. In all the lightweight concrete beams, higher the strength of the concrete the higher was the ultimate load carrying capacity.

ii. The contribution of oil-palm-boiler clinker (OPBC) in the OPSC beams increased the structural behaviour of the OPS concrete beams. The OPSC beams up to 35 grade showed a ductile behaviour, however, as the grade of OPSC increased the beam showed brittle failure without any warning. Whereas, the incorporation of 50% OPBC aggregate in OPSC significantly improves the structural performance and the failure nature of the beam.

iii. Contribution of the OPBC aggregate in OPS concrete significantly reduced the number of flexural and shear cracks at the of failure beams.

iv. The ratio of ultimate shear strength to the oven-dry density of the OPS-OPBC concrete beams was found about 36% higher than the strength to density ratio of the NWC beams.

v. Contribution of OPBC aggregate in OPS concrete showed better bonding with the cement paste which reduced the maximum crack widths of the beams at failure.

vi. All the beams showed pure shear failure, without any anchorage failure.

vii. The experimental results of ultimate shear strength of the beams were also compared with the predicted results. It was found that all the prediction models selected from different standards estimated very close results of the actual shear strength of the beams. On average, the ACI 318 and the EC 2 models predicted very conservative results of the ultimate shear strengths with the average ratio of 1.60.

10.2 Recommendations for Future

Further researches are required in order to encompass the different aspects of the behaviour of OPBC reinforced concrete beams. Following are some recommendations for future investigations and research.

i. Palm oil industry is producing millions of tons of solid waste materials (OPBC and OPS) every year, which may be are hazardous and producing environmental damage. The OPBC have good properties and suitable to use as structural material. So, the OPBC concrete can be studied in detail under different structural members.

ii. In general, OPBC lightweight has lower workability. Further study is needed to find proper mix proportions for this type of lightweight concrete with good workability.

iii. The long-term durability behaviour of OPBC concrete need further study. In addition, the properties such as drying shrinkage, creep, elastic modulus and bond strength are not defined extensively. Furthermore, research is needed to investigate the fire and sound resistance of OPBC concrete.

iv. Only few studies are available about the performance and properties of reinforced prestressed OPBC concrete beams under static loading. It can be further investigated in detail under both static and fatigue loadings.

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LIST OF PUBLICATIONS AND PAPERS PRESENTED

A. ISI Indexed Journals

1. **Muhammad Aslam**, Payam Shafigh, Mohd Zamin Jumaat. Oil-palm by-products as lightweight aggregate in concrete mixture - A review. *Journal of Cleaner Production* 2016, 126: 56-73.

2. **Muhammad Aslam**, Payam Shafigh, Mohd Zamin Jumaat, Mohamed Lachemi. Benefits of using blended waste coarse lightweight aggregates in structural lightweight aggregate concrete. *Journal of Cleaner Production* 2016, 119: 108-117.

3. **Muhammad Aslam**, Payam Shafigh, Mohd Zamin Jumaat. Drying shrinkage behaviour of structural lightweight aggregate concrete containing blended oil palm bio-products. *Journal of Cleaner Production* 2016, 127: 183-194.

4. **Muhammad Aslam**, Payam Shafigh, Mohd Zamin Jumaat. High Strength Lightweight Aggregate Concrete Using Blended Coarse Lightweight Aggregate Origin from Palm Oil Industry. Accepted in *Sains Malaysiana* 2016.

5. **Muhammad Aslam**, Payam Shafigh, Mohammad Alizadeh Nomeli, Mohd Zamin Jumaat. Manufacturing High-strength Lightweight Aggregate Concrete using Oil Palm Shell in a Semi-Lightweight Oil-palm-boiler clinker Concrete. *Revisions Received from Journal of Building Engineering,* 2016.

B. SCOPUS Indexed Journals

1. **Muhammad Aslam**, Payam Shafigh, Mohd Zamin Jumaat. Structural Lightweight Aggregate Concrete by Incorporating Solid Wastes as Coarse Lightweight Aggregate. *Applied Mechanics and Materials* 2015, 749: 337-342.

2. **Muhammad Aslam**, Payam Shafigh, Mohd Zamin Jumaat. Effect of replacement of oil-palm-boiler clinker with oil palm shell on the properties of concrete. *AIP Conference Proceedings* 2016, 1774: 030004-1 to 030004-6.

C. Papers Presented in International Conferences

1. **Muhammad Aslam**, Payam Shafigh, Mohd Zamin Jumaat. Structural lightweight aggregate concrete by incorporating solid wastes as coarse lightweight aggregate. *International Conference on Intelligent Materials and Manufacturing Engineering (IMME-2015)*, (Poster Presented), Jan 16-17, 2015, Phuket Island, Thailand.

2. **Muhammad Aslam**, Payam Shafigh, Mohd Zamin Jumaat. Effect of replacement of oil-palm-boiler clinker with oil palm shell on the properties of concrete. *International Conference on Advanced Science, Engineering and Technology (ICASET) 2015*: Lighting the Way Towards Sustainable, Penang, Malaysia.

D. Papers Ready to be Submitted in ISI-Indexed Journals

1. **Muhammad Aslam**, Payam Shafigh, Mohd Zamin Jumaat. Effect of coarse aggregate type on the drying shrinkage development of concrete.

2. **Muhammad Aslam**, Payam Shafigh, Mohd Zamin Jumaat. Prediction models estimations for drying shrinkage in oil-palm by-product lightweight concretes.

3. **Muhammad Aslam**, Payam Shafigh, Mohd Zamin Jumaat. Shear behaviour of oil-palm by-product lightweight aggregate concrete beams without shear reinforcement.