CHAPTER 1
INTRODUCTION

1.1 BACKGROUND

(a) Recycled aggregate

Concrete has been proved to be a leading construction material for more than a century. It is estimated that the global production of concrete is at an annual rate of 1 m^3 (approximately 2.5 tonnes) *per capita (*Neville 2003). The global consumption of natural aggregate (NA) will be in the range of 8-12 billion tonnes after 2010 (Tsung *et al.* 2006). Over 1 billion tonnes of construction and demolition waste (C&DW) is generated every year worldwide (Amnon 2004). The large-scale depletion of NA and the increased amounts of C&DW going to landfill sites are causing significant damage to the environment and developing serious problems denting the public and the environmentalist's aspirations for a waste-free society.

In the past, almost all materials which are used in the construction industry were entirely natural and all waste from demolished buildings was disposed of in landfills and partially in unauthorised places. The utilisation of the recycled aggregates created from processing C&DW in new construction has become more important over the last two decades. There are many factors contributing to this, from the availability of new material and the damage caused by the quarrying of NA and the increased disposal costs of waste materials. C&DW are generated mainly from demolished concrete and masonry structures. Due to advances in manufacturing of crushing machinery and recycling processes, it became possible to scale or crush down large masses of C&DW into smaller particles to produce recycled aggregate (RA) at acceptable cost.

The UK government is committed to sustainable development. Around 65 million tonnes of aggregate are derived annually from recycled sources representing 23.6% of the total of 275 million tonnes used annually in the UK (Wrap 2006). The government introduced the Aggregate Levy in April 2002. It is an environmental tax on the commercial exploitation of aggregate to address the environmental costs associated with quarrying that are not already covered by regulation, including noise, dust, visual intrusion, loss of amenity and damage to biodiversity. Many other countries, particularly those in the EU making increasing use of "green taxes", have introduced similar taxes to pursue environmental aims. The overall cost of landfill, particularly the

Landfill Tax, introduced over the last few years, is increasing. The tax is chargeable by weight and there are two rates: lower rate applies to those inactive (or inert) wastes; standard rate applies to all other taxable waste. The lower rate tax that applies to inert materials was set at £2 *per* tonne in 1996 then to £2.50 *per* tonne on 1 April 2007; no increase was made in 2008 and 2009. Table 1.1 shows increases of the lower and standard rates.

Table1.1 Increase of standard rate of landfill tax

Date	Rate (£/tonne)
1 Oct. 1996 to 31 Mar. 1999	7
1 Apr. 1999 to 31 Mar. 2000	10
1 Apr. 2000 to 31 Mar. 2001	11
1 Apr. 2001 to 31 Mar. 2002	12
1 Apr. 2002 to 31 Mar. 2003	13
1 Apr. 2003 to 31 Mar. 2004	14
1 Apr. 2004 to 31 Mar. 2005	15
1 Apr. 2005 to 31 Mar. 2006	18
1 Apr. 2006 to 31 Mar. 2007	21
1 Apr. 2007 to 31 Mar. 2008	24
1 Apr. 2008 to 31 Mar. 2009	32

The standard rate of landfill tax was increased to £24 *per* tonne; this applies to any standard rated disposal of waste made, or treated as made, on or after 1 April 2007; the legislation in Finance Bill 2007 stated that the standard rate would continue to increase by £3 *per* tonne in future years, on the way to a medium to long-term rate of £35 *per* tonne. The landfill tax rates was increased by 33% to £32 *per* tonne in April 2008 and the government intends to increase these by £8 *per* tonne each year so by the 2010/2011 financial year, a level of £48 a tonne will be reached - a 100% increase over 3 years. Organisations cannot afford to keep sending waste to landfill and need to implement recycling and waste saving initiatives as soon as possible in order to keep their costs down. As a result, the market for RA is expected to grow in significance within the construction industry during the coming two decades.

Unfortunately, BS EN 206-2:2000, the European standard for the specification of concrete did not include any provisions for the use of RA in concrete, however the UK complementary standard to BS EN 206, BS 8500-2: 2002 permits the use of RA but imposed additional requirements to be satisfied to allow RA in concrete such as those related to the maximum masonry content, level of fine materials, asphalt content, sulfate content, *etc*. Lack of codified provision does not, however, imply a prohibition on the use of RA.

In the past, recycled aggregates were used mainly in low utility applications such as general fill. Recently, these aggregates started to be used for intermediate utility applications such as foundations for building and roads. Nowadays, the aggregates are used, to a very limited extent, in high utility applications such as for the elements of buildings or structural layers of roads.

The advantages of recycling C&DW are numerous:

- Reduces the amount of C&DW entering landfill sites.
- Reduces the use of natural resources in construction.
- Contributes to the environment.
- Provides a renewable source of construction material.
- If used *in situ*, reduces haulage costs

The quality of the recycled aggregates has been improved significantly during the last decade as a result of good deconstruction practice and advances in stationary or transportable crushing machinery, as well as the recycling process itself *i.e.* screening and separation. As a result, improved quality aggregates are available nowadays, at prices competitive to NA. However, despite the enhanced quality of the recycled aggregates, the uptake of this alternative is still in fact too low (Dhir 2001; Wrap 2007). This limited use is largely due to the past experience formed when low strength cements and low quality recycled aggregates were used as well as the restrictions imposed by standards. Therefore, in view of current concreting technologies and advances in materials production, there is a need to reform the negative impression, prevalent for a relatively long time, to increase the use of recycled aggregates in construction. The most effective way is to increase the performance characteristics of the recycled aggregate concrete (RAC): that can be achieved by three mechanisms, chemical, physical and/or a combination of both. A possible option is the incorporation of other materials to substitute or supplement those conventionally used in concrete mixes. Mineral and chemical admixtures are among the materials which could lead to improved quality of RAC. The application of modern concreting techniques could result in improved performance.

(b) Pulverized fuel ash in concrete

Fly ash is a by-product of coal-fired electric power stations. Fly ashes are pulverized to produce pulverized fuel ash (PFA). PFA is a mineral known to contain high reactive silica content, and thus is an excellent pozzolanic material. Large amounts of ash are produced annually worldwide and present a real environmental challenge. The world production was 0.5×10^{12} t in 1989 (A2E03 2006), it reached 0.6×10^{9} t in 2000 (Kayali 2008). In Europe, statistics (ECOBA 2008) showed that about 64 million tonnes of ash were produced in 2004, out of this 68% was pulverized fuel ash (PFA) and nearly 22 million tonnes were utilized by the construction industry, for different purposes, this means that there is surplus of about 21.5 million tonnes. In Europe, more than nine million tonnes are used annually in cement and concrete (ECOBA 2009). Only about 25% of the global fly ash production is utilized (Rafat 2003a; Qin 2004). Fly ashes damage the environment when illegally dumped, occupy large volume when disposed of in landfill sites, and could be harmful to human and animal health. The recycling of fly ash in beneficial products would therefore be useful to alleviate its aforementioned adverse implications. Recycling fly ash in construction, including concrete, is a major approach to tackle this problem.

However, the use of PFA in construction is nothing new; it has a long history of use in concrete. More than seventy years ago, the US Bureau of Reclamation allowed the use of fly ash in construction (King 2005). Concrete containing a high volume of fly ash was mostly used in mass concrete *e.g.* roller-compacted dams and highway base courses (Poon & Wong 2000). In 1985, a high volume fly ash concrete for structural use was developed by the Canada Centre for Material and Energy Technology (CANMET), in which fly ash cement replacement level was used in many projects, typically at levels of 50 to 60% of cement content (Poon & Wong 2000). Today, the number of different construction projects with fly ash concrete throughout the world is on the rise. In some cases PFA replaces about 50% of the cement.

In the UK, PFA has been used in concrete for over 50 years. Due to the importance of PFA as a construction material, PFA was the subject of research for over 70 years with in excess of 10,000 papers being published (Sear 2005). National legislations and codes of practice were produced in many countries to simplify and expand its applications. BS EN 206-2:2000, and the UK complementary standard to BS EN 206, BS 8500-2: 2002 are standards that enable the use of fly ash as an addition to concrete. BS EN450, Parts 1 and 2: 2005, was produced to encourage the use of PFA by the construction industry.

PFA has some unique properties largely useful to a wide range of concrete characteristics. Most of the reasons for using PFA in any proportion in concrete are practical, to a large extent. In particular PFA in concrete offers the following advantages:

- Improves workability, hinders segregation and alleviates bleeding.
- Lowers heat of hydration and the risk of thermal cracks.
- Increases long-term strength.
- Reduces permeability and reduces the penetration of chlorides and sulphates as well as the potentials of corrosion of reinforcing steel.
- Enhances the concrete's ability to resist chemical attack and reduces harmful aggregate-silica reaction.
- Extends setting time and provides longer time for handling and casting of concrete.

Due to these merits, PFA nowadays is increasingly considered in all sectors of the concrete industry. However, the major uses of PFA in concrete were in NAC concrete, and the use in RAC concrete did not receive much attention, maybe due to the limited use of RAC itself. In this study, PFA will be used to partially substitute fine aggregate or cement to examine its influence on the performance of RAC concrete. When used to replace cement, PFA could, indirectly, reduce the 'high' CO_2 emission linked to cement production. RAC with PFA is obviously more sustainable than conventional RAC and NAC concrete.

(c) The use of superplasticizers in concrete

A wide range of superplasticizers (SP) are used in modern concrete technology. High strength natural aggregate concrete, in which only NA is commonly used, was proved to attain outstanding fresh and hardened properties, which are usually desired under particular exposure conditions. These may include, but are not limited to: high strength and stiffness, low permeability and good resistance to chemical attack. SP is often employed to aid the workability of concrete. Higher strength is produced when the water:cement (w/c) ratio is reduced to the amount needed for cement hydration. Without SP, the concrete mix's w/c ratio can not be reduced that much as this usually results in a stiff concrete of low workability (Neville 2003). To obtain the most effective level of SP content required to improve the performance characteristics of concrete, the type and the optimum dosage of SP must be determined early in the design stage by concrete trial mixes. Alternatively, manufacturer recommendations may yield good results, even though trials are strongly recommended to account for material property variations. However, the compatibility of SP with the binding materials and its influence on the rate of slump loss are major concerns. Whenever a new SP or a proven SP is to be used with a new binder, compatibility must be ensured to avoid negative results. In this study a new generation of superplasticizer (SP) will be researched.

1.2 RESEARCH OBJECTIVES

For more than one reason, the concrete community need to appreciate that projects should go in harmony with the concept of sustainable development. Therefore, the use of recycled aggregates in construction, to the maximum possible limit, is becoming a necessity more than a desire. One of the most likely feasible ways is to use recycled aggregates in the production of RAC. Chemical and mineral admixtures that exhibit pozzolanic behaviour, such as PFA, have been used to produce natural aggregate concrete (NAC), particularly high strength concrete (HSC). The effect of these materials on the behaviour of NAC is very well established, but there has been little or no attention given to their use and effect on RAC. More specifically, it is believed that not enough has been done so far to increase the use of RA to replace NA in concrete construction, particularly in higher-grade applications. However, the following are the objectives of this research:

- To prepare a comprehensive literature review that encompasses the sourcing, production, and use of RA in concrete. The literature will also include the role of fly ash and superplasticizers.
- To study the influence of polymer-based new generation SP and PFA produced to new standards, on the properties of RAC and to investigate its fundamental properties. Mixes with SP and mixes in which PFA partially replaces fine aggregate and cement will be examined.
- To examine the potential of recycling the red granite dust (RGD) in new concrete.
- To examine the potential of producing RAC concretes made with SP, PFA, and RGD in both conventional and self-compacting concrete (SCC).

The successful production of RAC, with the help of these materials, could lead to large-scale use of recycled aggregates for various structural applications, rather than its prevailing low value uses. This is deemed possible when the strength and performance of RAC is improved to compete with NAC.

1.3 GOOD QUALITY RAC

It is well known that concrete is formed of three components namely the aggregates, the cement paste and the interfacial zone between coarse aggregate and the paste. When the aggregate to be used is limiting the strength of concrete, the overall performance is usually affected; in this case, the obvious solution is to improve the performance of the aggregate and other concrete components. Aggregate strength may be enhanced by blending lower quality aggregate with a better quality one, while the strengths of the matrix and the interfacial zone can be enhanced when certain techniques are adopted. In this context, the benefits offered by PFA, and other pozzolanic binders, and SP can be utilised.

It is believed that when RAC mixes are designed as normal concrete mixes with w/c > 0.5 and higher slumps, it is highly unlikely to be able to attain adequate strength, therefore SP and PFA will be used in this study to produce good quality RAC, in a similar way to which higher strength NAC concrete is usually made. A 100% RA will replace NA in all concrete mixes (except in one mix in which 50% RA will replace NA to investigate the effect of blended aggregate). Good quality RAC would mean a concrete made with a 100% coarse recycled aggregate, produced from demolished concrete structures, that achieves a 28 day compressive strength above 50 N/mm^2.

1.4 SIGNIFICANCE OF THE STUDY

For economical, environmental reasons, and due to the increased amount of recycled aggregates nowadays as a result of advances in crushing technologies, there has been a growing global interest in maximizing the use of recycled aggregates in construction. In view of the increased volumes of C&DW and industrial by-products such as fly ash and RGD (more details about the scale of C&DW and fly ash are given in Chapter 2) as well as the advantages offered by the use of admixtures in modern concrete, it is considered very beneficial, from different prospects, to utilise these abundant materials in producing good quality RAC concrete; the type of concrete having similar performance characteristics to NAC. When proved successful, RAC can be substituted for NAC in many concrete applications.

Most of the knowledge and experience with RAC arose from the use of concrete components of lower quality when compared with current materials. Concreting techniques are also improved significantly. Therefore, research in this area could lead to an added value to our knowledge and widen the use of these wastes.

1.5 STRUCTURE OF THE THESIS

The thesis is structured as shown in Fig. 1.1. The thesis consists of nine chapters summarized as follows:

- Chapter 1: presents an introduction to this study and briefly provides basic information about C&DW, RA, SP, and PFA in concrete. The environmental problems caused by the disposal of these materials and the advantages of utilizing these wastes in new concrete were briefly discussed. The objectives and the significance of the study were clearly stated.
- Chapter 2: presents the motivations of the study, and briefly reviews some important issues which were considered essential prerequisites to everyone dealing with recycling and recycled aggregate in concrete. The need for recycling and the scale of construction waste were also presented and discussed. The processing of C&DW was also introduced; Bowhill Aggregate Recycling facility in Fife, UK, from which the recycled aggregate used throughout this study was supplied, was focused upon. The factors contributing to increased performance, on the basis of previous experience, were suggested.
- Chapter 3: presents a literature review for RA and fly ash in new concrete. A brief background on recycling of old concrete and its impurities, which usually exist in recycled aggregates, is given. Differences between recycled and natural aggregates and classification of recycled aggregates were also extensively reviewed. The review of previous research on the properties of RAC and on the utilisation of PFA in natural aggregate concrete are given a considerable part of this chapter; finally, the advantages of using these materials are discussed and summarised and conclusions were drawn.
- Chapter 4: in this chapter, the main hypotheses and the methodology used in this study were introduced. The materials used in this study were described and their characterisation and test results were presented and discussed. This chapter also describes the way an SP was selected, on the basis of compatibility with the concrete components, for further research work. The compatibility was examined through detailed testing of the setting times, water demand

characteristics of the binder paste and the loss of slump of the concrete mixes. The selection process of a suitable reduction of the mixing water (and therefore the w/c ratio) to match the chosen SP was also clearly explained.

- Chapter 5: the influence of SP on the performance characteristics of RAC concrete was presented in this chapter. A review of some important aspects related to testing of hardened concrete and modes of failure was briefly reported. The results of the experimental work were reported and discussed.
- Chapter 6: in this chapter, PFA was used to substitute fine aggregate at different levels, and the effects on RAC concrete's fresh and hardened performance were reported and evaluated. Comparison was made against similar NAC mixes.
- Chapter 7: this chapter is divided into two parts. Firstly, the effect of substituting 30% of cement with PFA in superplasticized mixes was extensively studied. Secondly, the possibility of recycling red granite dust powder (RGD), by partial substitution for cement in NAC and RAC concrete, was examined. The optimum level of substitution was evaluated on the basis of fresh and hardened properties. Concrete mixes using the selected level were produced, tested, reported, and the results discussed. Due to the encouraging results obtained in Chapters 5 and 6, the work in this chapter was extended to include other fundamental properties of concrete including flexural strength, modulus of elasticity and non-destructive testing *i.e.* the pulse velocity of concrete. Comparisons between fly ash and red granite dust concrete were made. The determination of the properties of these concretes with the models exists in standards and pertinent literature was evaluated.
- Chapter 8: presents results of testing the potential of using RA to produce self-compacting concrete (SCC). The requirements and testing methods used to ensure production of this type of concrete were reviewed and reported. PFA and RGD were used to produce the RAC-SCC concrete.
- Chapter 9: The conclusions of the study and recommendation for future research are presented.

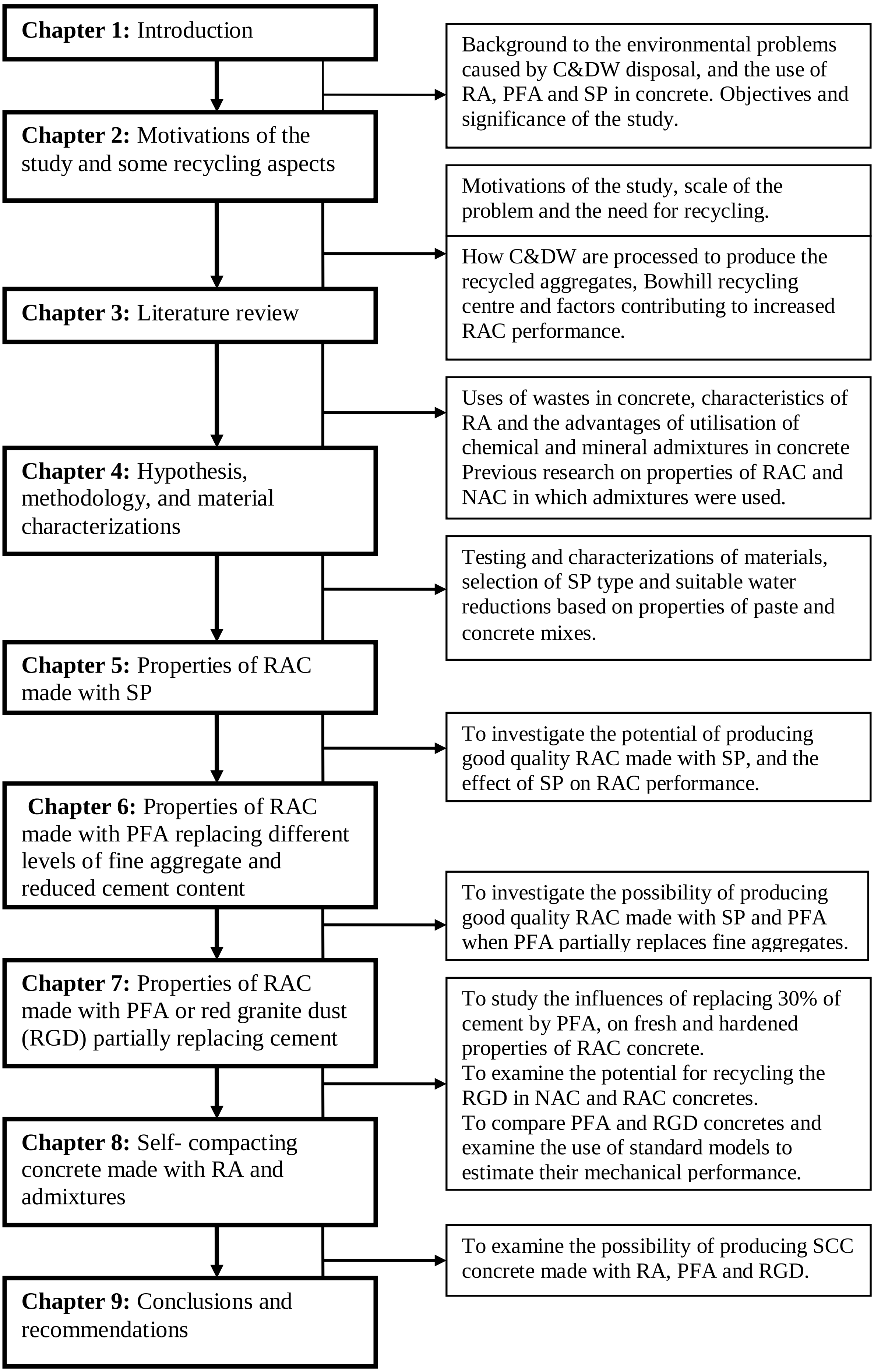


Fig. 1.1 Thesis structure

CHAPTER 2
MOTIVATION FOR THE RESEARCH: RECYCLING AND CONCRETING ASPECTS

2.1 BACKGROUND

The data in this chapter were considered an essential prerequisite to everyone involved in the recycling of concrete waste. Much herein may be widely known; a briefing will be provided here to set the scene.

Recycled aggregates are those resulting from the processing of materials previously used in construction (Wrap 2007). Recycled aggregates can be broadly subdivided into two main categories: RA derived predominantly from crushed concrete rubbles and RA created from the rather broad field of C&DW such as brick-based RA and asphalt-based RA.

The level of impurities in the second category (particularly those derived from asphalt pavements) is usually medium to high and can significantly affect the strength and performance when recycled in concrete; therefore aggregates in this category are often used for secondary applications and are of little interest for use in concrete. Another barrier facing the use of this category in concrete are the limitation and provisions set in standards such as BS 8500-2: 2002. In this standard, restrictions were put on the maximum masonry content, fines content, asphalt, acid-soluble sulphate, and other contaminant material such as glass, plastics, and metals. In addition, rejection by the public and the concrete industry, particularly in places where there is a good reserve of proven natural aggregate has been a barrier. With these restrictions, it is most unlikely that concrete suppliers would be able to accept this type of RA for concrete mixes.

Therefore, this study will focus on the utilization of RA created mainly from crushing old concrete masses, the type of RA that contains little or no impurities and produced in a recycling plant; and from now on RA would mean aggregate in this category. However, there are also some restrictions on this type of aggregate: many standards, guidelines and codes worldwide specify 20% RA replacement for NA in concrete (more details will be presented in Chapter 3) for a number of reasons. However these barriers have contributed to formulating resistance and the negative image prevalent for the last two decades in minds of the engineers and contractors about the use of RA in concrete.

The performance related issues are always cited as the major concern leading to the limited use of RA in concrete, however, technologies are advancing rapidly and it is time when the performance of RAC should be enhanced to maximize the use and value of RA in concrete construction. The first essential step in this direction is to identify possible sources of weakness of RA in terms of material properties; these are summarised in Fig. 2.1.

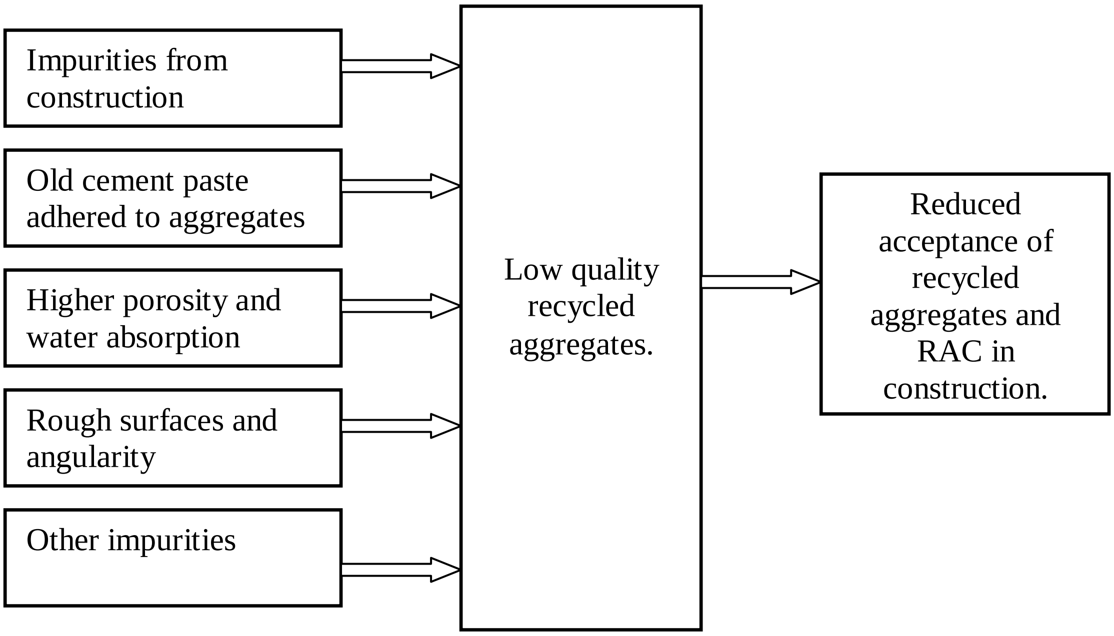


Fig. 2.1 Sources of weakness of RA

When materials with these properties and contents are used, it will significantly influence concrete performance and discourage the use of RA in concrete. Treatment of RA to offset part, or all, of them will make much better aggregate. Rough surface is considered a source of weakness as it has significant effect on RAC's workability, however the mechanisms of how these sources lead to decreased RA quality will be explained throughout this thesis.

In this chapter, ideas about the need for recycling and current technologies will be presented. The scale of C&DW and fly ash will also be presented. The RA which will be used in this study was supplied from Bowhill Recycling Facility, therefore, a short description of the facility, the equipment used and its recycling process will also be provided here, to give the reader the necessary detail underpinning current recycling technology.

2.2 SCALE OF WASTE MATERIALS AND THE NEED FOR RECYCLING

(a) General

Waste materials and by-products are created from several sources. This study will focus on waste arising from construction materials and fly ashes. When structures are demolished, the resulting material is either dumped in landfill sites or recycled in new applications. There is always a need for innovation, rehabilitation, refurbishment, demolition, and rebuilding. These activities will result in C&DW. Rejected concrete and concrete materials not satisfying code requirements or due to delayed casting are also creating C&DW. Therefore, C&DW can be considered a renewable material. This leads to the simple four-step closed-loop lifecycle of materials used in construction as shown in Fig. 2.2.

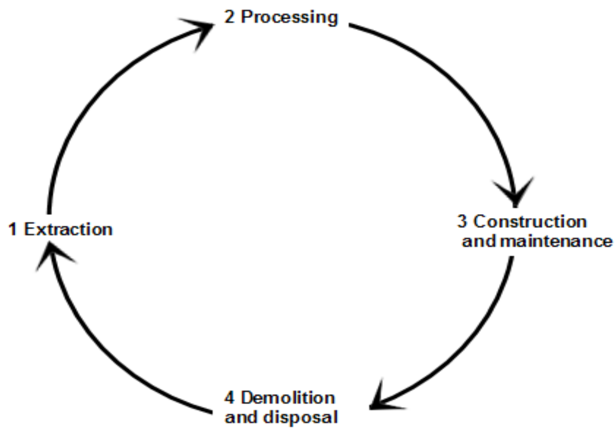


Fig. 2.2 Life cycle of construction materials

Fig. 2.3 shows a sample of buildings under demolition in Edinburgh, UK.

Fig. 2.3 Samples of buildings under demolition in Edinburgh

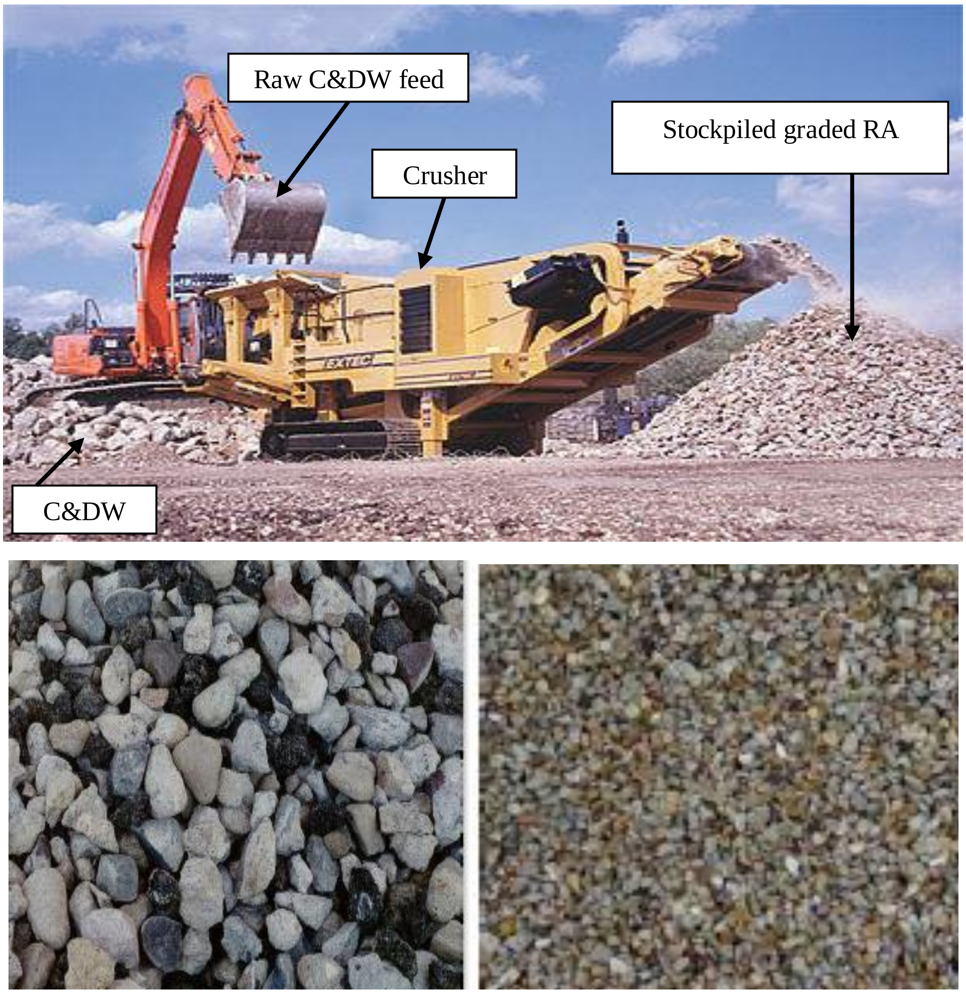
Over the last few years the environment has become a major concern worldwide putting more pressure on the need for an increase in waste recycling and a reduction in disposal to landfill sites. Under the Kyoto protocol, 85% of waste will be recycled by 2013. The amount of waste that was going to landfill in the UK was 85% of the total, however, in 2001 the Land Fill Directive was introduced to improve standards and reduce negative effects on the environment. The Landfill Tax Regulations were also introduced in 1996 as an incentive to reduce the amount of waste going to landfill sites and encourage it to be recycled instead. The avoidance of landfill dumping charges for C&DW is an economic incentive that supports the reclamation of recycled aggregates in construction (Gilpin *et al.* 2004).

The main cause for concern was the environmental impacts that natural aggregate extraction is having; the loss of usual countryside landscape, visual disturbance, noise, dust, increased traffic and blasting sound and vibration are the major problems associated with aggregate extraction. To minimize these problems while causing the least possible hindrance to operating and meet aggregate demand, several researchers (Sherwood 1995; Speare 1995) have suggested that more use should be made of waste and recycled materials.

Excellent quality aggregates such as granite, limestone, basalt, dolomite, *etc*. are used unnecessarily for many low value applications or for those with low strength requirements when a number of recycled aggregates can be used instead. Natural aggregates are in short supply and have already become exhausted, in many places worldwide; new sources of aggregate for different construction projects were required. Therefore, there is a need for further development and research into the utilisation of recycled aggregates for use in higher-value applications including structural concrete elements. The properties of RAC concrete must be improved to meet the performance requirements and criteria of modern concrete.

(b) Construction and demolition waste

The amount of C&DW is increasing worldwide due to population growth and burgeoning construction activities. In contrast to natural resources, C&DW can be considered a renewable source of materials because there is always a need for innovation, rehabilitation, refurbishment, demolition, and rebuilding. The trend or the way of life common nowadays worldwide is that all kinds of wastes need to be turned into environmentally friendly materials; every possible measure has to be taken to put them to beneficial uses. Fig. 2.4 shows C&DW being processed to make recycled aggregates of different sizes.



(a) 20 mm RA (b) 10 mm RA

Fig. 2.4 Turning C&DW to recycled aggregates of different sizes

Fig. 2.5 The variability in shape, texture, and origin (seen simply by colour) of the RA

The latest available data on waste arising published by the Office for National Statistics, UK (ONS www.statistics.gov.uk) showed that the UK generated an estimated 333.4 million tonnes of waste in 2002/2003 (published in 2005). Fig. 2.6 shows that C&DW accounted for 107.5 million tonnes.

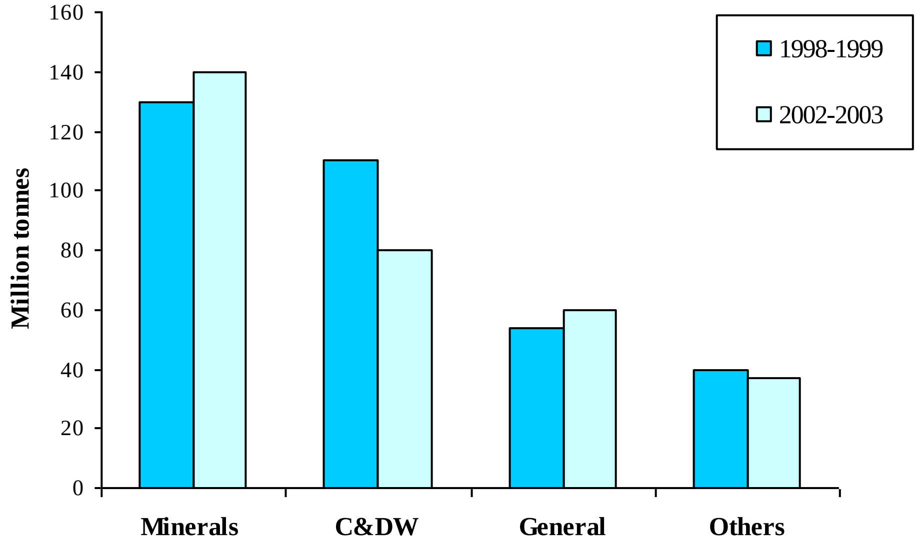


Fig. 2.6 Waste arisings by type in the UK (ONS 2005)

Fig. 2.6 shows that mineral and C&DW together accounted for 71% of all UK waste, therefore, they are the major sources of waste materials. However, even though the data presented here date back to 2005, it is predicted to be of the same pattern for the years 2005 to now (2009) *i.e.* wastes generated from quarrying of minerals and C&DW remain the major portions. The implication of this inference is that recycling has not taken off as much as it should have.

Similarly, Fig. 2.7 was reproduced from the most up-to-date data *i.e.* 2007-2008 published by the Department for Environment, Food and Rural Affairs (Defra 2009), it displays the estimated annual waste arising by sector in the UK, in which the total arisings were about 335 million tonnes and of this 32% are C&DW materials accounting for more than 107 million tonnes, thus the main sources of waste are the construction industry. When combined with wastes from mining and quarrying, industrial, and dredged material the figure rises to 80% of total arisings. A quantity of this size is indeed worrying and alarming.

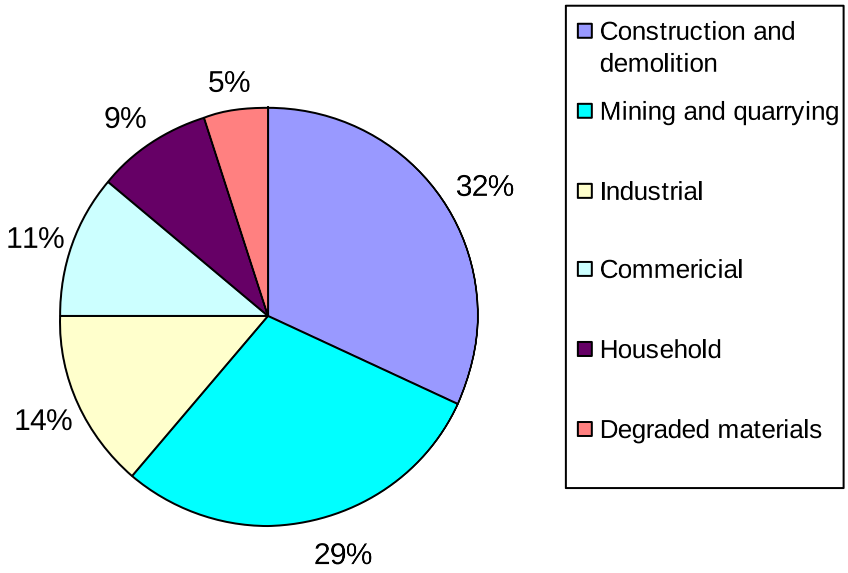


Fig. 2.7 Estimated annual UK arisings (Defra 2009)

It was reported that about 500 kg *per capita* C&DW was produced in the European Union (Nik 2005) and therefore the construction industry is responsible for creating 50% of total waste, consumes 40% of energy and takes 50% of the produced natural materials. Substitution of coarse aggregates of a size more than 4.75 mm *i.e.* coarse aggregates was recommended by this study.

Around 200 million tonnes of natural aggregates are produced annually in the UK (ONS, UK) of which 55% is used by the construction industry. The annual production of C&DW in the UK is approximately 110 million tonnes and approximately half of the produced C&DW is currently recycled and used as a source of aggregates (Wrap 2007). These figures indicate the sheer scale of C&DW; the volume of C&DW is equal to that of the aggregates used in construction. In, 1993, the Construction Industry Research and Information Association (CIRIA 1993) estimated that about 10 million tonnes of construction waste were recycled in the UK, mainly for low-grade applications such as fill materials, whereas only 1.1 million tons of concrete waste were being crushed, graded, and added to new ready-mixed concrete each year. This value was expected to increase to 17 million tonnes by 2001 and 24 million tonnes by 2011 (McLaughlin 1993). High-grade utilization such as structural concrete has been discouraged by the lack of suitable specifications (Collins 1993).

In Scotland, the most recent published Scottish Aggregate Survey (SAS 2007) (this survey is often published every four years) showed that the amount of aggregates was

33.5 million tonnes in 2005 of which 24.7 million tonnes were crushed rock and 8.8 million tonnes were sand and gravel. The overall contribution from recycled and secondary aggregate has reached only 18%. This trend is expected to continue for the foreseeable future as the production was steady over the period 1990-2005. SAS also reported that around 5 million tonnes is likely to be exported. However, although these figures appeared to be economically viable as natural aggregates are readily available at acceptable prices, the environmental implications are unfortunately negative; this simply means more depletion of natural resources, damage to natural landscapes due to quarrying, increased disposal of C&DW and more landfill and disposal sites having to open.

Owing to the age of the vast majority of current infrastructure worldwide, in which most are concrete structures and increased demand on new-build, the amount of C&DW is predicted to increase and hike the construction material prices considerably in years to come. A recent study (Crawford 2007) showed that UK house prices have risen more rapidly than in any country in Europe. The study showed that the average cost of a home increased by 90% compared with an average of 40% in Euro-zone countries in the period 2001 to 2006. This, however, should lead to increased demand for natural materials in Scotland, increase their cost and therefore encourage recycling.

Other European countries are facing practically the same problem; it is estimated that the annual generation of C&DW waste in the EU could be as much as 450 million tonnes; if the earth and some other wastes were excluded; the construction and demolition waste generated is estimated to be 180 million tonnes *per* annum (Akash *et al.* 2007). An EU study calculated that an average of 28% of all C&DW was recycled in the late 1990s (European Commission Report 1999). Most EU member countries have established goals for recycling that range from 50% to 90%. Recycled materials are generally less expensive than natural materials, and recycling in Germany, The Netherlands and Denmark is less costly than disposal (Akash *et al.* 2007). The situation is even more serious in less developed countries, and areas with fewer natural suitable construction materials. Sustainable development is pushing towards developing effective techniques to deal with these problems. There are no easy options, but recycling in new concrete is likely to be the best. This investigation is, however, an attempt to produce good quality recycled aggregate concrete.

In the home country of the author, Libya, the need to start recycling is becoming urgent. Libya was kept under UN sanctions for political reasons for the last two decades. The impacts on infrastructure, facilities, and the entire construction sector were significant. The government suspended maintenance and new construction plans for long periods, except some strategic projects such as the Great Man-made River, certain railway lines and the like. Construction activities in other sectors were kept to a minimum in size and budget allocated for their implementation during the sanctions.

In Libya, a number of infrastructure projects were built during the Italian colony (1911-1970) in the major cities most of which are located on the Mediterranean coastal region. In addition to deterioration due to marine climate, their original proposed design life must be over by now. The other considerable part of Libya's infrastructure was constructed during the country's first construction boom from 1970-1980; however due to a lack of, or inappropriate, maintenance programmes, climatic conditions and high cost of construction materials, many structures have been damaged, severely in some cases. As a result it is estimated that at the very least 50% of existing structures need heavy rehabilitation and refurbishment; 15-25% should be demolished soon.

However, the case is different nowadays; UN sanctions were lifted in 2003 and government is strongly committed to development, particularly in the construction sector. Libya is experiencing another construction boom and has remained relatively unscathed by the global financial crisis. Demolition is the predominant scene in all Libyan cities and towns in 2007, 2008, and 2009 generating a huge amount of C&DW.

There is severe shortage of all kinds of buildings such as houses, schools, hospitals, offices, hotels, *etc.* and large quantities of construction materials are needed. The General Planning Council of Libya advised the government to allocate at least 60% of every year's budget for infrastructure development in successive plans (2008-2025) (Libyan information centre 2008). During the last five years, the rapid infrastructural development and growing demand for housing has led to shortage and rises in the cost of construction materials.

Most waste materials produced by demolished structures are disposed of by dumping in landfill sites or in unauthorised places causing environmental problems. In very limited cases C&DW were used to improve the quality of poor local sandy roads in rural and countryside farms. Libya has a huge reserve of construction material, particularly all kinds of good quality aggregate, rocks, quartzitic fine aggregate and large open unused spaces where C&DW can be safely dumped. However, the annual produced C&DW is estimated in the range of 400-450 kg *per capita*; volumes are too large to accommodate. The transportation cost will press developers to dump in landfill sites located inside urban areas and there will be a space problem in addition to the environmental concern.

Therefore, it is necessary to start recycling and re-use of demolition concrete waste to save environment, cost, and energy.

Most of Libya's construction resources are available however in the northern part of the country. Recycling will be an effective option for the construction industry, particularly in the remote desert areas which suffer a lack of construction material needed for different purposes, especially roads, in addition to the higher cost of transport. However, recycled material will not find easy acceptance by client and construction industry, but at least it can be used for secondary jobs when enforced by appropriate legislations.

The quantity of concrete discarded every year has reached a staggering figure and is estimated to double or more within 10-15 years. The preceding discussion leads one to conclude that there is a desperate need to maximize the use of RA derived from C&DW in construction, particularly in new concrete, and therefore, the performance of RAC needs to be improved to satisfy the requirements of the concrete industry. This in turn will contribute to reduced energy consumption, conserved natural resources, and make concrete a more "green" material.

(c) Fly ash

Fly ash is another post-industrial waste going to landfill sites. Like C&DW, fly ash is presenting problems, if not properly controlled. The most serious is the damage to the ecosystem; predominantly the atmosphere and air quality (air pollution) and the contamination of groundwater as the size of fly ash grains are sub micron-size and can easily percolate to groundwater, thus posing a public health risk.

Statistics showed that fly ash is produced in huge amounts, for instance, in Europe; about 43.5 million tonnes of PFA were produced in 2004. Fig. 2.8 shows the use of PFA in the construction industry to be nearly 22 million tonnes for different applications (ecoba 2006). In 2006, the United States produced nearly 125 million tonnes of coal combustion products, comprising fly ash**,** flue gas de-sulfurization (FGD) materials, bottom ash, boiler slag and other power plant by-products (American Coal Ash Association 2006) of which 43% were used beneficially, and nearly 70 million tonnes were disposed. In China, it was reported that the annual discharge of fly ash was up to 0.27 billion tonnes and the cumulative available is 2 billion tonnes. The annual global production of fly ash is about 6×10^8 tonnes out of which 20 to 25% is utilised in the construction industry (Shafiq 2006). The total volume to 2020 is expected to be doubled and the total cumulative available to rise to 3 billion tonnes (Gao *et al.* 2007). In India, over 70% of electricity is generated by combustion of fossil fuels, of which nearly 61% is produced by coal-fired plants resulting in about 100 million tonnes of ash annually (Aggarwal & Gupta 2007), while the utilisation is not more than 15% (Rafat 2004).

Similar trends have been reported elsewhere in many countries: global production of cement is estimated at 2.7 billion tonnes in 2007 and should reach 3.4 billion tonnes and 3.5 billion tonnes by 2010 and 2012 respectively; demand is expected to grow by 4.7% annually (Rock Products 2009; Bharat Book Bureau 2009). Greater dependency on fossil fuel such as coal for electricity generation due to the current global economical crisis could result in more production of fly ash. The latest data, published in February 2009 (www.ecoba.com) showed significant increase of greenhouse gas emissions because developing countries, particularly China and India, shaw a massive surge in electric power generation based mainly on coal burning, thus resulting in more fly ash. The trend would therefore continue if more countries turned to using coal for their energy production. Previous experience with fly ash in cement-based products and concrete proved to be economically viable and an eco-friendly choice.

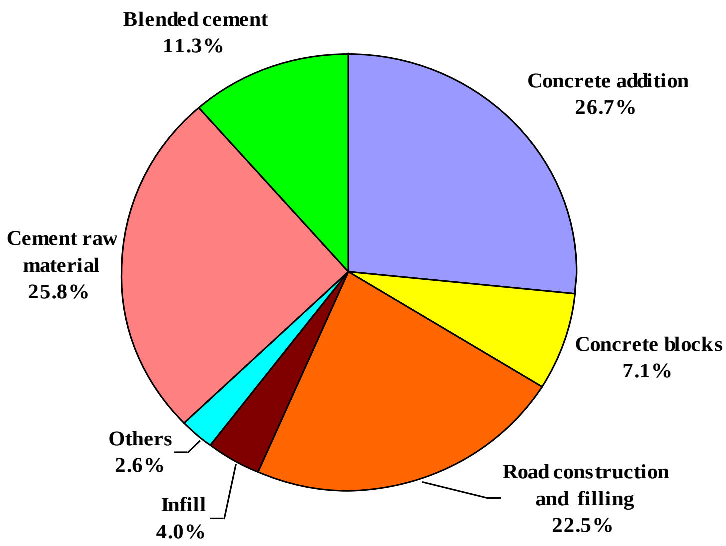


Fig. 2.8 Utilisation of PFA in the construction industry in Europe in 2004 (Ecoba 2006)

It is very well known that an increase in cement content of a mix generally increases concrete strength; this approach is typically adopted by many design methods. However, this could result in increased fines and water contents; particularly for mixes with high cement content *i.e.* rich mixes. Cost is also consequently increased. Alternatively, similar quality concrete can be achieved by keeping the cement content fixed while reducing the mixing water while using chemical admixtures. Another approach is to replace cement partially by one mineral admixture or more in combination with low water content and SP. PFA is a mineral admixture that exhibits pozzolanic activity (a pozzolana is a natural or artificial material containing silica in reactive form, so-named after Pozzuoli in Italy).

It was mentioned earlier that the use of mineral admixtures such as PFA in concrete offers many advantages and results in cost savings, particularly when these materials are diverted away from dumping sites. The substitution of PFA in concrete reduces the amount of cement, improves concrete properties, reduces the energy needed for processing natural materials, and conserves natural resources (Malhotra 1990). The reduction of cement production reduces CO_2 emissions and energy consumption (Suneel 2004).

As the amount of waste materials increases, more volume is occupied at landfill sites. There are over 4000 landfill sites in the UK; each year approximately 111 million tonnes of controlled waste (household, commercial and industrial waste) are disposed of in landfill sites (Waste 2008). If landfill dumping is continued at current rate, the UK will need to open many new landfill sites.

Fly ashes are widely used nowadays in producing ready-mixed cement. In this type of cement, PFA is mixed as a dry base with cement in different percentages (in most cases < 30% PFA), to form what are widely known as pozzolanic cements, and packaged for general construction applications. The quality of pozzolanic cement is not different from ordinary Portland cement (Massazza 1993). Pozzolanic cements showed: lower early strength, higher later ultimate strength, lower heat of hydration, and lower permeability.

d) Red granite dust

Red granite dust (RGD) is the fine powder produced from the rock faces when rocks are cut and crushed to produce coarse aggregate. Large amounts of RGD were accumulated as waste powder in quarries during the production process causing local health difficulties and space problems for many quarries in Scotland. Considerable amounts of this waste material go to landfill sites; quarry waste is estimated using a waste to saleable product ratio equal to 1 to 9; that is one tonne of quarry waste is generated for every nine tonnes of quarry products (British Geological Survey 2009). Large amount of by-products are produced annually in the UK; Fig. 2.9 shows the estimated total amounts of mining and quarrying by-products in the UK, 1990-2004.

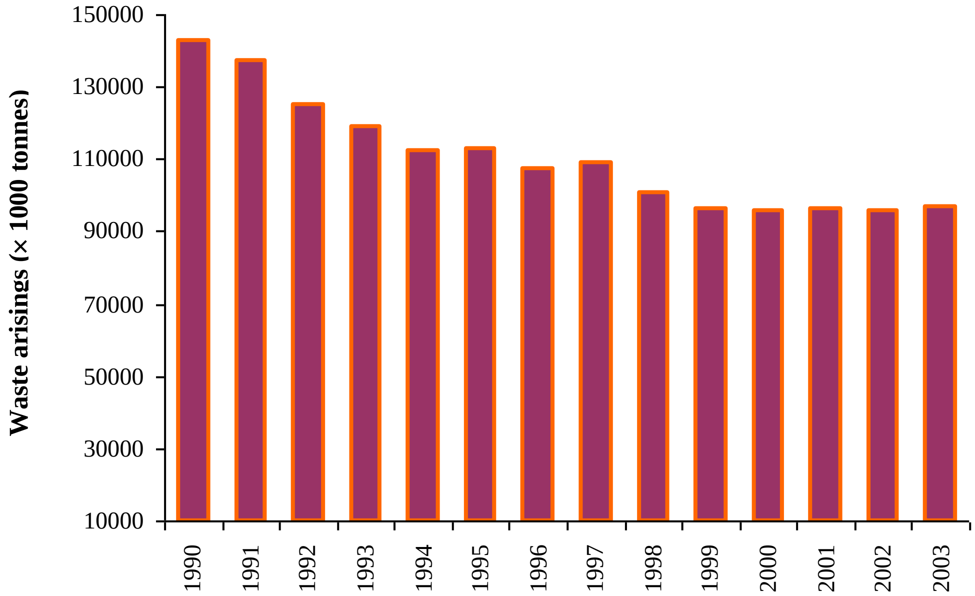


Fig. 2.9 Estimated total quantities of mining and quarrying by-products in the UK (1990-2004) (British Geological Survey 2007).

2.3 PROCESSING OF C&DW MATERIALS

C&DW crushing and processing equipment and their correspondent accessories used to make RA are similar to those commonly used for production of crushed aggregate from natural rock. It is not the aim of this study to discuss in detail the processing techniques, machinery and controlling devices, or processing methods. However, in general processing plants incorporate various types of crushers, screens, transfer equipments (conveying belts), magnets for separation of ferrous material, then sorting devices for dry and wet removal of other substances from crushed concrete. For more details, reference can be made to relevant resources such as the US. Environmental Protection Agency (www.epa.gov). Recycling equipment has received considerable coverage over the last two decade and advances are ranging from simple to very sophisticated (www.epa.gov). Portable and fixed facilities are readily available in the market at accepted cost; the number of recycling enterprises is increasing worldwide. Fig. 2.10 shows a modern fixed recycling centre in Hong Kong; the major sections of the facility are displayed.

In general, recycling plant basically comprises: a primary sorting facility, crushers *i.e.* primary and secondary, impurity removal facilities (magnetic tools to separate metals, air knives, and manual sorting belts), removal services (heavy duty trucks and tractors), stockpiles, and storage areas.

Fig. 2.10 Fixed recycling facility (Fong & Jaime 2002)

C&DW materials are crushed to produce granular materials of smaller particle size. The intended application of the recycled material and the impurities content are the main factors controlling the degree of processing required. Fig. 2.11 shows a flow chart detailing how C&DW is processed (DeVenny 1999).

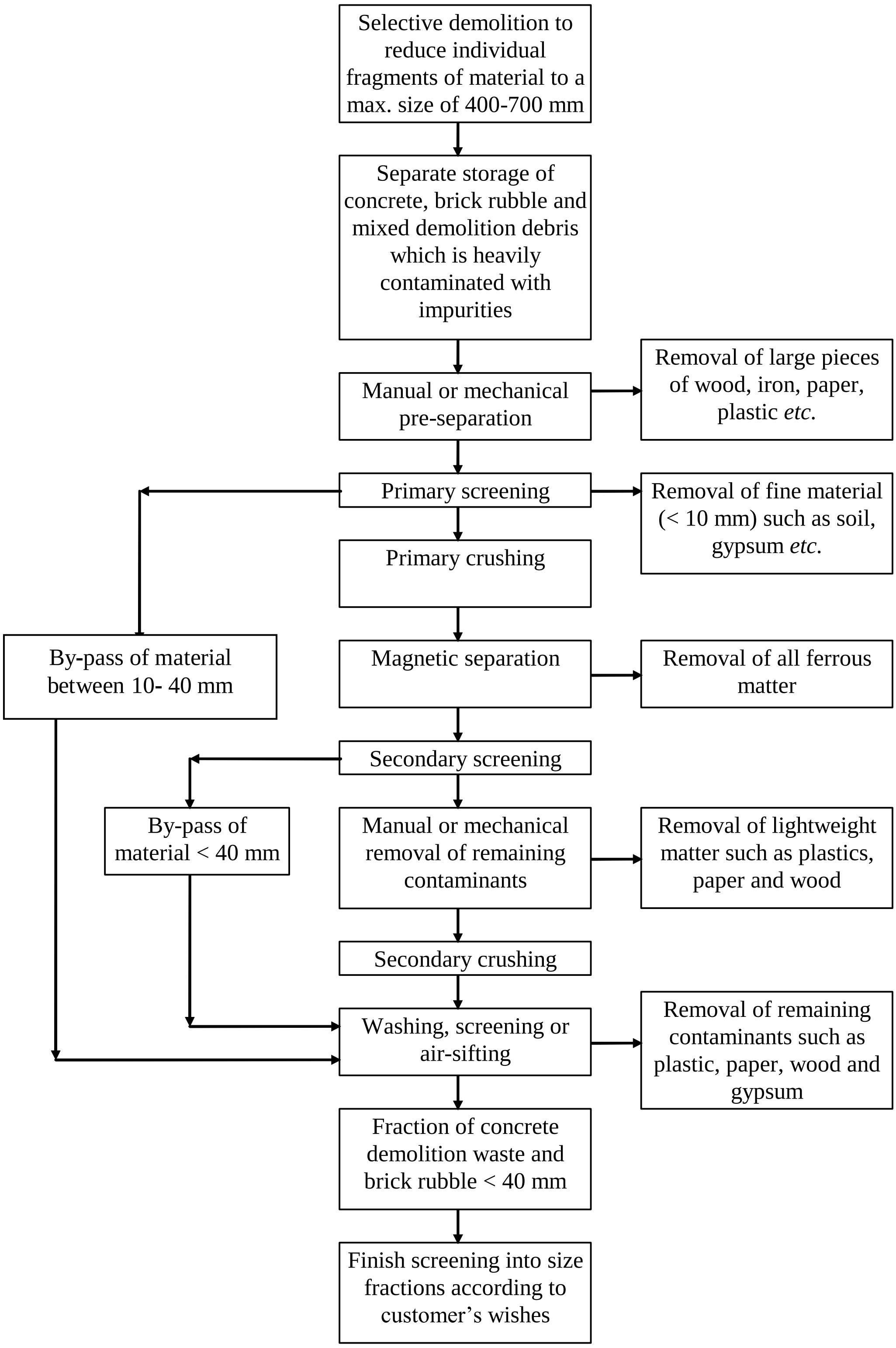


Fig. 2.11 Processing of demolition wastes (Devenny 1999)

(a) Bowhill aggregate recycling facility

Because the RA used in this study was delivered from Bowhill, Fife, Scotland, a brief description of the facility and its recycling equipment will be given here. The aim is to give the reader an overview of the recycling process in which new technologies are used. As part of this study, several site visits were arranged and representative samples from the aggregate stockpiles obtained for use throughout this study.

However, there are other recycling plants in business across Scotland and the UK. The number is increasing. Bowhill facility has been in operation since 2003. It is operated by Realm Construction Ltd. The site was developed in three phases from 2003 using grants supplied by the Waste and Resources Action Programme (WRAP), which receives the funding from the Scottish Executive. In Phase 1, the machinery for screening and washing was installed; this included a Powerscreen MK2 conveyor and magnet. This allowed 60% of the material entering the site to be recycled.

In 2004, Phase 2 was completed with the installation of another power screen, a crushing machine and a stock piler. The material entering this stage included anything smaller than 40 mm from the first stage. With the two stages combined, 85% of material entering the site was being recycled. Phase 3 was finished in 2006 and the recycling of 95% of all material entering the site was achievable by installing four-bay picking stations, a Powerscreen Finemaster 60, and a high speed Tonne Bagger.

The site also recycles the water used in these operations by using reed beds on site to remove the gully waste. The clean water is collected in an open pond and then pumped to storage tanks. From here it can be used in the aggregate cleaning process or to fill tankers that are used for keeping dust down around the site. Due to these phases Realm Construction are aiming to produce 361,000 tonnes of washed RA over a five year period.

(i) Types of equipment in Bowhill recycling facility

There is a variety of equipment used to complete the whole process of recycling material on any recycling site. Some of the machines used on the Bowhill site are shown in Figs 2.11 and 2.12. The Kue Ken crusher is used to crush material as large as 250 mm down to 60 mm before screening. It is capable of crushing 50 tonnes *per* hour depending on how far down the crusher is closed.

The Powerscreen Mark 2 is used to grade the material into different sizes using screens. The material is placed into the hopper where any oversized material is discharged using a reject grid. From here the material falls onto a conveyor belt into more screens of different grades, where the material is fully separated into different piles.

These are very useful pieces of equipment as they can be moved easily around the site and can also be transported to the demolition site. This would be very useful if there is a considerable amount of building rubble to be recycled and if the distance is too great to transport all the rubble from the demolition site to the recycling plant. This can also be beneficial as the recycled material can be reused on the construction site instead of buying material. CO_2 emissions associated with expensive road haulage are also avoided as are indirect congestion-related environmental costs.

Fig. 2.12 Aggregate screen

The other main machines on the Bowhill site include the Powerscrub 120R (Fig. 2.13) which is used to wash the aggregate free of any dirt and contamination. It uses scrubbing as a method of cleaning; by working the material off the material itself it removes dirt and clay. It also contains two water baths for soaking the material and a final rinse area to ensure the aggregate is fully cleaned. The other machinery on this site includes loading shovels, track excavators and dump trucks. These are all involved in the piling and transporting of material around the site.

Fig. 2.13 Powerscrub 120R crushing machine

(ii) Recycling process at Bowhill facility

At the Bowhill site the recycling process is as follows:

1- C&DW is placed into the Powerscrub for washing and then it moves onto the Powerscreen for grading. Any metal in the rubble is removed using conveyors and magnets so it can also be recycled. Material larger than 40 mm goes straight to the stockpile while the rest goes to stage two.

2- The crushing of the aggregate takes place here followed by further screening of the material. Filtered water is introduced at this point to wash the aggregates again so that no fines or silt residue are present. From this stage stockpiles are made using washed 40 mm, 20 mm, 10 mm and 5 mm aggregate sizes. The fines are removed in the washings to stage three (Fig. 2.14).

3- This is where the fines are placed in the Powerscreen Finemaster 60. It is able to remove any silts and clays from the sand so that it satisfies the specifications required for building sand. This material can also be bagged at the site using the high speed tonne bagger.

Fig. 2.14 Removing fines by washing with water

Fig. 2.15 shows a sample of coarse RA created at the Bowhill facility, for comparison reasons another sample of coarse NA (red granite used in this study) is shown in Fig. 2.16. Clearly the RA sample is less homogenous and contains some contaminants.

Fig. 2.15 Bowhill produced 20 mm coarse RA

Fig. 2.16 Red granite 20 mm NA

More recently (Kiyoshi 2007) suggested a new production method for recycling aggregate for concrete. The RA is produced by a simple assembled system of equipment with the possibility to be mixed with NA for better quality. Kiyoshi believed that by adjusting the mixing ratio, the required quality of concrete can be ensured. Fig. 2.17 displays the proposed new facility. RA produced in recycling facilities under strict quality control measures was reported to exhibit better properties than RA from other sources.

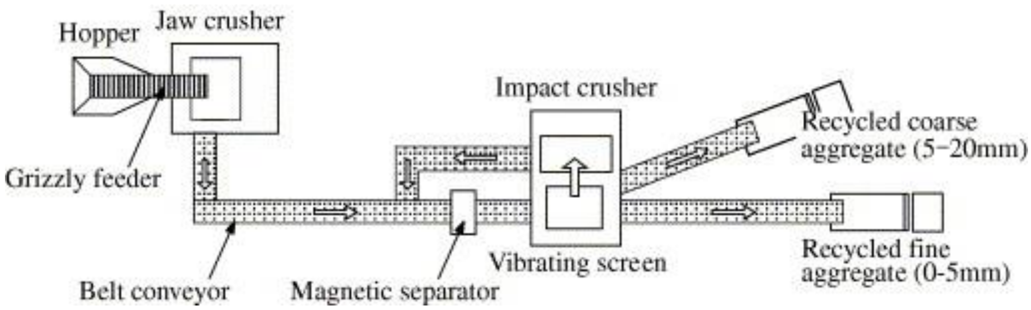


Fig. 2.17 Production plant for recycled aggregates (Kiyoshi 2007)

Recycling leads to many environmental and economical advantages, but there are also recycling-related costs, which should be paid to satisfy the modern society's environmental requirements. However, the use of RA in construction should be encouraged even if NA is available at better price and performance. On the other hand, recycling could also imply disadvantages; the potential for vibration and noise due to recycling, moving and hauling machinery, aesthetic issues, dust, and the congestion caused by trucks transporting C&DW in normal traffic. The proper selection of the location of the recycling plant and the good management of the facility, whenever these disadvantages are considered, can ameliorate their possible impacts.

2.4 FACTORS CONTRIBUTING TO STRENGTH AND PERFORMANCE OF RAC

It is crucial to know the factors that lead to enhanced RAC quality. The factors that contribute to increased strength and performance are summarised in Fig. 2.18.

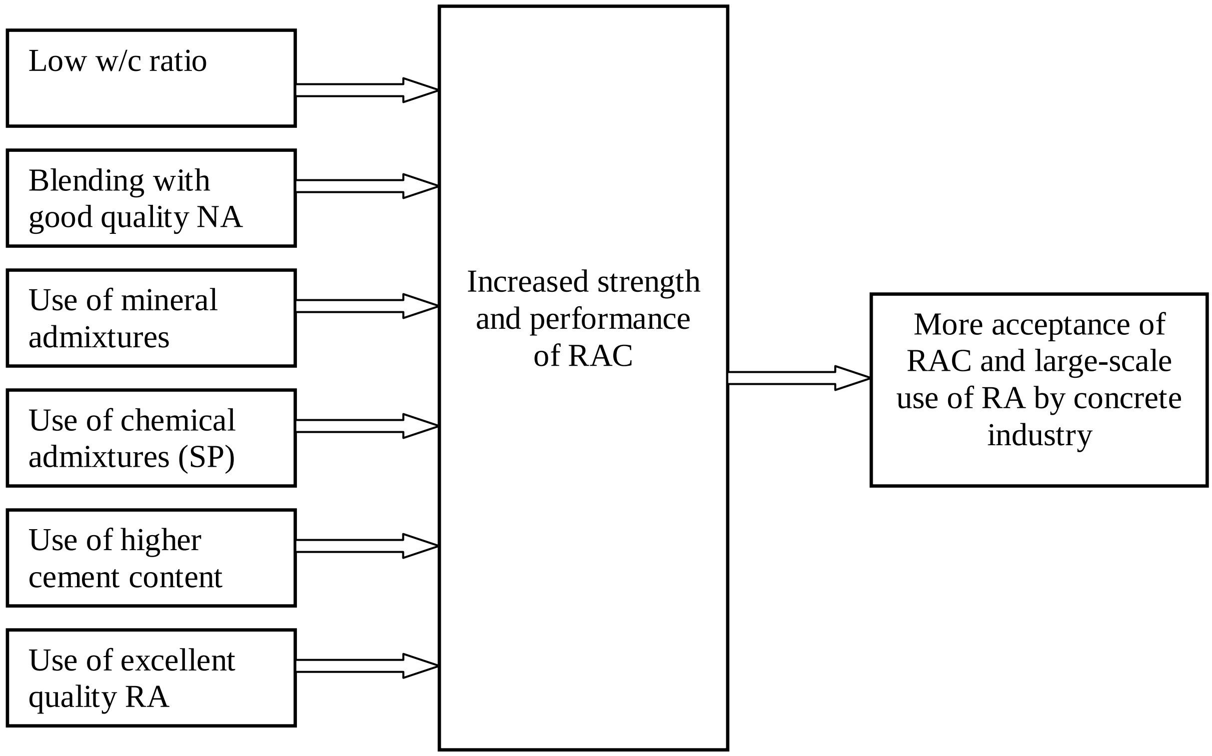


Fig. 2.18 Factors contributing to increased RAC strength and performance

Brief details of the role of these factors are described as follows:

(a) Water to cement ratio

It is well known that the major factor controlling the strength of concrete is the water-cement ratio (w/c) or more precisely water to binder ratio (w/b). The more water in a concrete mix the lower the quality at all ages. The w/b has very significant effects on both fresh and hardened properties of concrete. Strength and durability are considerably reduced when w/b ratio is increased; the less strong, the less durable the concrete. The effect of w/b ratio on the fresh properties of concrete restricts the choice of a low value on the strength, but SP can effectively remedy this situation.

Although higher water content will certainly make the concrete more workable, but with a higher w/b ratio; excess water will leave voids inside the concrete mass and will result in a weak paste, particularly in the transition zone; the very thin layer between the aggregate and the paste near the concrete cover zone. The voids and the transition zone are commonly, particularly in low grade mixes, where microcracks are initiated and propagated to decrease concrete strength. In general, concrete with a low w/c ratio is stronger and less permeable, thus more durable as compared with high w/c mixes. Therefore, water should be controlled right the way through batching, mixing, transport, placing and curing to ensure good quality concrete. The mechanism of influence of w/c on many properties and performance characteristics is extensively studied and can be found elsewhere.

(b) Blending RA with good quality NA

Aggregates form a considerable volume of concrete; typically 60-70% by volume. In addition to its serving as cheap filler, aggregate contributes to the strength of concrete, particularly when the strength of the paste matrix is low. When a concrete section is overloaded, cracks start to form and propagate across the aggregate grains; cracks are significantly reduced or stopped whenever encountered by strong aggregate. Soft, friable aggregates are not capable of resisting even lower stress developed across their mass and, therefore, are not expected to produce good concrete. Previous experience showed that when the quality of RA is low and limiting the ultimate strength of concrete, blending of these types of aggregate with a proven quality NA will result in improved quality aggregate mixture, and better resultant concrete.

(c) Use of mineral admixtures

Mineral admixtures which are primarily characterised by high quantities of reactive silica, such as natural volcanic ashes and tuffs, fly ashes, ground granulated blast furnace slag and other highly active pozzolans like silica fume, rice hull ash and metakaolin, were proved to be beneficial to concrete. When water is added to cement, a gel of calcium silicate hydrate (CSH) and hydrated lime (CH) are formed. CSH is the glue that holds everything in concrete together and gives concrete its strength and durability. CH in contrast is brittle crystals that gather on the surface of the aggregate particles and rebar making a weakened matrix. When a source of additional reactive silica (such as fly ash) is added, it will react with CH in presence of moisture to form more CSH. More details about the PFA reaction are given in Chapter 3.

(d) Use of chemical admixtures

The use of chemical admixtures in concrete began in the 1960s and was a milestone in concrete technology (Malhotra 1997). Previous research findings demonstrate that the use of fly ash and chemical admixtures, which often result in reduced water contents in the mix, can be beneficial to concrete performance, and this generally applies across the whole range of properties, including aspects of durability, *i.e.* heat of hydration and drying shrinkage (Dhir 1998). The use of chemical and mineral admixtures improves the pore structure of concrete, provides additional reduction to the porosity of the mortar matrix and enhances the interface with the aggregate (Hassan 2000). However, in general, a higher rate of slump loss of superplasticized concrete was also reported by many researchers especially when traditional sulfonated naphthalene formaldehyde base admixtures are used (Dhir *et al.* 1998; Liu *et al.* 2000). Most of the research gave cognisance to the importance of the compatibility of superplasticizers with the binding materials.

Chemical admixture SPs are known as water-reducers which are commonly used in modern concrete. SPs are known either as superplasticizers, superfluidizers, superfluidifiers, super water-reducers or high range water-reducers. The basic advantage of SP is to enhance the workability of concrete at low w/c ratio, thus it makes concrete mixtures very workable and suitable to cast in congested reinforcement and inaccessible areas, complex shuttering and deep sections, floors, foundation slabs, bridges, pavements and roof decks. Pumping of concrete is very much improved by incorporating SP in the mixtures.

The reduction in mixing water when SPs are used results in a much denser cement matrix than in ordinary concrete; as mixing water is limited, cement grains grasp all the available water to complete the hydration process. As a result, there would be no excess water to be entrapped in the concrete to produce voids or to migrate to its surface (bleeding) to form microscopic ruts and, therefore, both high quality cement paste and chemical bond are achieved which eventually leads to higher compressive strength.

(e) Use of higher cement content

Cement is the binding agent or the glue that holds the other constituents of concrete together. Cement is known to be the most expensive component of concrete. Concrete contains cement typically in the range of 7-15%. When extra cement is added to a mix, more calcium silicate hydrate (CSH) is produced; however beyond a certain level, depending upon water available for hydration, a fraction of the added cement may not be hydrated and works as fine filler resulting in denser concrete. In addition to higher cost, excessive cement addition usually results in more heat, particularly in thick sections or large mass concrete pours leading to shrinkage and thermal cracks. However the addition of up to 10% cement will increase the volume of the paste and improve workability, strength, and durability of concrete without negative impacts; the reduction of cement content, to a level that does not affect performance, therefore results in some other benefits.

2.5 THE NEED FOR MORE RESEARCH WORK ON RAC

In view of the aforesaid figures and discussion, it can be predicted that there will be a large reserve of C&DW worldwide and therefore considerable amounts of RA are going to be available in the near future. The main reasons for the increase of this material are:

- Many old buildings and structures have exceeded their useful life and need to be demolished. For instance, 62% of England's housing stock was built after 1944 (ONS 2009). Many are scheduled to be demolished and new housing fit for the 21st century is in either planning or construction. For instance, in the UK 140,000 properties are planned to be built in 2009; the government's target of building 3 million new homes by 2020 (National Housing Federation, UK) is still with us regardless of its political, social, or economic validity.
- Large scale earthquakes, landslides and war-devastated areas often result in huge amounts of destruction; new safety requirements necessitate demolishing even those mildly, or partially-damaged structures.
- The volume of concrete production is growing worldwide, which in turn will result in a continued expansion in the extraction of natural aggregates and the deterioration of the environment and virgin landscape of the countryside.

The obvious alternative source of aggregate is the recycling of C&DW materials, but unless the performance of recycled aggregates is improved, the use of RA in construction will not be increased, and therefore the contribution to address the aforementioned problems will be inadequate. A recycled aggregate that satisfies the requirements of good aggregate is expected to produce better RAC, while those containing soft, friable particles, or contaminated by harmful substances such as dust, wood, asphalt, alkalis, or sulphates will result in concrete of low strength, and reduced durability.

The assessment of the problem size above demonstrated that C&DW and post-industrial wastes, particularly fly ash, are the most abundant materials readily available as potential substitutes for cement, fine aggregate, and natural aggregates in urban areas. Both are produced in large amounts in many places around the world, and frequently end up in landfill sites presenting a challenging environmental problem. In view of these findings, the need for more research work is obvious to match society's requirements for economic and safe disposal of wastes. However, all parties such as policy makers, planners, developers, designers, contractors, and suppliers need to be involved and should play a part to make the practical use of these research findings possible. The effective use of these wastes (and of course others) is considered a key component of global construction ecology.

In view of the aforesaid challenges, C&DW should be recycled. The use of recycled aggregates and other industrial by-products in new concrete should be practically possible when it can be clearly shown that such materials can successfully produce concretes having equivalent quality to NAC concrete. In other words, when it is proven possible to create RAC concrete capable of achieving design strengths, in addition to cost and durability-related requirements, its industry-wide acceptability will have been achieved. In such case, there should be no reason to reject the use of this type of concrete instead of NAC in several concrete applications. The UK concrete industry has recently shown commitment to responsible sourcing by being the first construction products sector to sign-up for BRE Environmental and Sustainability Standard BES 6001 (BRE 2009).

This research will investigate the possibility of recycling these materials in new RAC concrete. In this perspective, the following questions could be raised: is it possible to increase the amount of RA in making RAC if new materials such as modern polymer-based superplasticizers and mineral admixtures, complying with the latest published European standard, are utilised? If the answer is most likely to be yes, then what is the best level of replacement of RA? What is the best dosage of an SP? What are the most favoured mineral admixtures and what is the best substitution or supplement level? What are the influences they could have on the properties and the impact on performance of the end product? These questions, however, form a new point of research. Research on RAC can highlight unclear issues and build or increase the confidence to use RAC concrete and widen its applications.

CHAPTER 3
LITERATURE REVIEW

3.1 INTRODUCTION

Sustainable development is defined by Bruntland's report to the World Commission on the Environment (Our Common Future 1987) as; "Development which meets the needs at present without compromising the needs of future generations to meet their own needs." This statement is the central theme, which the UK government have incorporated into the White Paper on the environment: "This Common Inheritance (CM 1200; HMSO September 1990)" and "Sustainable Development: the UK strategy (CM 2426; HMSO 1994)" (1994). Great importance has therefore, been placed upon the issue of sustainable development.

The main target of sustainable development is to address the consumption and management of natural mineral resources, whilst minimising the effects that our working environment has on these resources. The Sustainable Development Strategy gives advice on the minimisation of waste and encourages efficient use of natural minerals. It outlines proposals for government, local authorities and the construction industry to promote and recycle waste materials, which could be substituted for primary minerals extracted from natural resources. This will involve researching work previously completed in this area and review the results achieved.

This chapter presents a review of previously completed work on the use of RA to make new recycled concrete (RAC), the literature review will focus on the performance characteristics of this type of concrete. Concrete in which PFA was employed will also be extensively reviewed, as it is intended to use this material to enhance the properties of RAC. However, more background materials will be introduced here and the engineering /scientific material will follow.

3.2 THE USE OF RA IN NEW CONCRETE: A HISTORICAL PERSPECTIVE

The investigation on recycling of concrete waste was initiated by Glushge in Russia in 1946, and in the following years, a number of experimental investigations have been carried out (Xio 2006). It was reported (DeVenny 1999) that large scale use of RA derived from C&DW in concreting has been noticed after the Second World War in Germany and to a lesser extent in the UK. The recycling was necessary to meet the massive demand for building materials to rebuild war-torn cities. During the war, a large amount of devastated and severely damaged structures had to be demolished and the product reused in different applications. In Germany, a total of 11.5 million m^3 of recycled materials was produced by the end of 1955, with which 175000 dwelling units were built (DeVenny 2005). In other parts of Europe, recycled aggregates had been in use for different primitive applications at that time. (MacNeil 1993) contends that using recycled and secondary materials will fill the gap between supply and demand of these finite resources.

The use of RA in new concrete (RAC) is restricted by standards and codes of practice, worldwide. For instance, in the UK, a protocol is in effect for recycling RA in new concrete permitting the use of up to 20% by mass. In Hong Kong, it is legal to use 100% coarse RA in proportioning low grade concrete (grade 20 or less), but only 20% by mass in high grade concrete (Nik 2005). In the Netherlands, the construction national organization permitted in 1994, the use of RA and admitted that the use of 20% by mass RA results in little difference in the properties of fresh or hardened concrete (DeVries 1996).

Governments around the globe are dedicated to increasing the use of RA in construction, for instance, the aim of recycling RA created from C&DW material in concrete has been set at 40% since 1991 (Nik 2005) in Germany. In Japan, the Government began implementing a program in June 2000 called “The Basic Plan for Establishing a Recycling-Based Society” providing a 10 year period to promote comprehensive and systematic policies aimed at changing unsustainable patterns of production and consumption (www.env.go.jp). In the USA, many federal and state highway contracts specify the use of recycled materials in highway construction. Recycled aggregates are also specified for housing construction; for bedding, capping, piling mats, below foundation layers and sub-base for footpaths. Recycled aggregates were used in concrete and asphalt pavements to build new roads and their associated concrete works. (www.epa.gov).

BS 8500-2: 2002 provides a basis for the use of RA in concrete. However, to date statistics showed that only a small proportion of recycled aggregates is used in high utility applications (Wrap 2007) and that is probably due to existing provisions and limitations. Very recently, BS EN 12620:2002 + A1:2008, aggregates for concrete, was published to supersede BS EN 12620:2002. It specifies the properties of natural aggregates, fillers and manufactured or recycled aggregates for use in concrete. This standard incorporated few clauses for recycled aggregates, however, it was reported in this standard that new test methods for recycled aggregates are at an advanced stage of preparation.

A number of successful projects have recently been reported, in which recycled aggregates were used in a new concrete, for instance, in the UK the use of Demolition Protocol was adopted in the Wembley Development in London (Wrap Report AGG0078 2006). Works at Wembley started in 2004 over a 15 year timescale. In this project, a demolition recovery target of 80% was specified. It is estimated, that 5,670 tonnes of concrete and masonry material would be reprocessed on site for the new-build, and 26,163 tonnes of recycled aggregate will be used for engineered fill applications of the new-build. In Scotland, demolition can be seen everywhere, particularly in the major cities, for instance deconstruction works started in 2008 for regeneration of Sighthill areas in both Glasgow and Edinburgh, in which 19-storey residential complexes built in 1967 will be demolished and replaced by new houses (that both regeneration sites are called Sighthill is purely coincidental).

A number of housing development and small scale projects, in which recycled material was partially used are constructed worldwide. Relevant examples are: in the UK, RAC for housing development projects in North Bracknell, the demolition and reconstruction of an 8-storey Plymouth office block, the deconstruction and rebuild of the Twickenham South Stand and the new national stadium (Wembley). The construction of a new high school in Sørumsand, outside the city of Oslo, Norway in 2001 in which RAC had been used; 35% of natural coarse aggregate was replaced by RA derived from old concrete to construct some elements such as its foundation, basement walls, and columns. However, it is outwith the scope of this chapter to give a comprehensive list of projects in which RA was used, several cases are published elsewhere.

Literature also showed that good quality natural aggregates were extensively used unnecessarily for a variety of cement based components; the following lists such applications:

- Building masonry bricks and blocks.
- Paving blocks and road kerbs.
- Cast *in situ* and pre-cast concrete paving units and slabs.

- Non-structural concrete walls and partitions.
- As a pipe bedding material.
- For capping, piling mats, and sub-base layers in housing development.
- For road sub-base sidewalks and footpaths.
- For soil improvement, gabions, erosion control, general and engineered backfilling.
- For producing low grade concrete for different concrete structures.

Instead, considerable amounts of natural aggregate could have been saved if suitable recycled aggregates were used for these applications.

3.3 IMPURITIES IN RECYCLED AGGREGATES

Contaminants are the major factors which limit increased use of recycled aggregates in the concrete industry. Recycled materials will vary depending on their origin. Unless removed, contaminants present in the original material will be passed on to the new concrete, and could have detrimental effects on strength and durability. To obtain good quality materials from recycling sites, many researchers (Buck 1977; Collins 1993; Bairagi & Kishore 1993; Vivian & Tam 2007) suggested that contaminants should be removed before crushing. This is best done at source where materials such as glass and timber can be removed. After crushing, other contaminants can be removed off a conveyor belt (by being hand-picked) or by use of a large magnet. As *per* BS 8500-2:2002, limits were imposed on recycled aggregate composition as shown in Table 3.1.

Table 3.1 Requirements for recycled aggregates (BS 8500-2:2002)

Property	Percentage (by mass)
Maximum masonry content	10.0
Maximum fines content	3.0
Maximum lightweight materials content (density < 1000 kg/m^3)	1.0
Maximum asphalt content	10.0
Maximum other foreign materials (*e.g.* glass, wood plastics, metals *etc*.)	1.0
Maximum acid-soluble sulphates SO_3	1.0

However, after October 2003, BS 8500-2:2002 was republished incorporating Amendment No. 1 and these limitations were removed. RA users need to assess the suitability of material on a case-by-case basis.

The most common contaminants usually found in recycled aggregates are now discussed.

(a) Mortar

Recycled aggregates are composed of the original aggregates and the adhered mortar. It is well known that physical properties of recycled aggregates are very much dependent on the type and quality of the adhered mortar. Quality of recycled aggregate is dependent on the type of aggregate present in the demolished concrete or brick and the type of mortar used. Cement mortar is much more difficult to remove than lime mortar, so aggregates that have this mortar adhered to them are usually crushed to aggregate. It is therefore inevitable that crushed masonry aggregate will have a considerable loose mortar content compared to aggregates derived from crushed concrete. However, lime mortar can be removed at source to leave good quality aggregate behind.

The crushing procedure of the RA and the size of recycled concrete masses have an influence on the amount of adhered mortar (Hansen 1985). It is believed that the impact crusher removes much more adhered mortar than other methods. The adhered mortar is a porous material; its porosity depends upon the w/c ratio of the recycled concrete employed (Etxeberria *et al.* 2007). Cement mortar adhered to aggregate is a source of micro-cracks and reduced bond. Cracks in the adhered mortar due to crushing can be considered as a source of weakness (Bashar & Ghassan 2008). However, with a high strength matrix, cracks can be filled with new mortar to appreciably increase the matrix-aggregate bond.

(b) Bitumen

Bituminous materials are obtained from crude petroleum. There are three types of bituminous mixes, which are used in road construction: penetration grade bitumen; this bitumen is used to bind aggregate whilst hot. Cut–back bitumen; this is a blend of penetration grade bitumen mixed with a volatile oil, *e.g.* creosote, and is normally used for surface dressing. Bitumen emulsions; these emulsions are made of globules of bitumen in water with added emulsifiers. Emulsions are normally used to bind the layers of a road.

Bituminous materials and aggregate covered in bitumen create a poor bond with cement. The bitumen tends to peel away from the cement. Bituminous materials present in recycled aggregate will greatly reduce the strength of the resulting concrete. (Hansen & Narud 1992) found that the percentage increase of bitumen present in recycled aggregate is proportional to a decrease in the resultant strength of the concrete. It was concluded that by adding 30% asphalt into concrete mixes caused the resulting concrete to lose 30% of its compressive strength. This study also concluded that there were no obvious reasons why stringent limits were not imposed upon the allowable contents of bituminous aggregate particles even though the strength reduction is evident.

Bituminous materials have a general effect of reducing concrete strengths in a similar way to the effect of low-strength lightweight aggregate on ordinary concrete (Sagoe 1998). In BS 8500-2:2002, bituminous materials (termed as asphalt) were limited to 5% by mass, while the recently published BS EN 12620:2002+A1:2008 allows the use of recycled aggregates containing a maximum bituminous materials content of 10%.

(c) Gypsum

Water-soluble sulphates in recycled aggregates that incorporate gypsum plaster are potentially reactive and may give rise to expansive disruption and damaged concrete (known as sulphate expansion). Some codes (in the mildest area) limit the amount of gypsum content in aggregate to 0.5% by weight of the aggregates or 0.4% by weight of the cement including sulphate in cement. (Hansen & Narud 1992) reviewed several studies of the deleterious effects of gypsum plaster presented in recycled aggregates used to produce RAC. From these studies it was concluded that strict limits on gypsum content should be included in standard specifications for recycled aggregates. Recommendations suggest applying the above mentioned limits or alternatively, using sulphate resistant Portland cement for the production of RAC concrete where the recycled aggregate may be contaminated with gypsum.

However, work (Dhir 2001) showed that there were no concerns with expansion resulting from use of gypsum-contaminated RAC provided the gypsum content was sufficiently low; that is the total sulphate content of the aggregate is less than 1% by mass. In some places such as The Netherlands, demolished wastes containing any gypsum are considered contaminated, and are not used to produce recycled aggregates. Tests on recycled aggregate samples obtained from different sources have been shown to give mean acid-soluble sulphate contents between 0.2 – 0.7% by mass; this range is well below the limit of 1.0% set in BS 8500-2:2002 (Dhir & Paine 2003; Wrap 2007).

Recently (BS EN 12620:2002+A1:2008) gypsum is classified in the family of "other constituents" and given the designation X; recycled aggregates may be specified for RAC if the content of all other constituents X (including gypsum) and glass R_g are not exceeding 1% by mass; although stricter limits of 0.5% or 0.2% by mass may be applied.

(d) Organic materials

Many organic substances such as paper, wood, plastic, textile fabrics, joint seals, and other polymeric materials are unstable in concrete, particularly when subjected to drying-wetting or freeze-thaw cycles (DeVenny 1999). The amount of lightweight materials (particle density less than 1000 kg/m^3) was limited to 0.4% by mass in BS 8500-2:2002 (for highway specifications). However, literature showed the content of these materials could be substantially reduced when they are removed from building sites before demolition.

When material goes to C&DW, fragments of wood and plastic (the largest usual contaminants from buildings) are often difficult to segregate; it is good practice to separate them by air knives, and/or by hand (larger pieces) from a moving conveyor belt before crushing, or between primary and secondary crushers. Floating material in BS EN 12620-2:2002+A1:2008 (designated as FL) is allowed to a maximum of 5%, although stricter categories (as small as 2% or 0.2%) may be specified. Soil and clay are often sticking to coarse grains and if not removed, then their presence in recycled aggregate concrete could be deleterious (Wrap 2007). The usual requirements for cleaning may be applied to specification; the option of washing the recycled aggregate to remove these materials before crushing was effective, although a large amount of water is frequently used. Organic content in RA may degrade with time and create voids in the concrete's microstructure (Bashar & Ghassan 2008).

(e) Chlorides and sulphates

It is well established that the presence of chlorides, sulphates and other salts in natural aggregates have little significant influence on the properties of plain concrete but in reinforced concrete they can give rise to corrosion of steel reinforcement. This also applies for recycled aggregates. If sulphates are present in sufficient quantities they can react with cement compounds when RAC concrete is produced; leading to excessive expansion and ultimately the deterioration of hardened concrete in presence of moisture.

Concrete standards such as BS EN 206-1:2000 impose a limit on the chloride content of concrete between 0.2 - 1.0% by mass of cement based on the sum of the contributions from all concrete components. In a study (Wrap 2007), the acid-soluble chloride content was measured in specimens of recycled aggregates obtained from three different sources; results have shown that chloride contents were 0.02, 0.04 and 0.2%. The use of marine or river fine aggregates in the original concrete or brick can possibly give rise to acid-soluble chloride content in recycled aggregates (Wrap 2007). Two categories were included in BS EN 12620-2:2002+A1:2008 for recycled aggregates based on water-soluble sulphate (SS) content; these categories are: (i) $SS_{0.2}$, with a limit on water-soluble sulphate content of 0.2% by mass, and (ii) a category with no requirement (SS_{NR}). Recycled aggregates could also contain organic materials or other substances in proportions that alter the rate of setting and hardening of concrete. Water-soluble materials from recycled aggregate could therefore influence the initial setting time of RAC concrete; BS EN 12620-2:2002+A1:2008 defines four categories for the influence of water-soluble material from recycled aggregates on the initial setting time, based on the change in initial setting time of the cement paste, the change in this time was ≤ 10 minutes for category A_{10}, ≤ 40 minutes for category A_{40}, > 40 minutes for category $A_{Declared}$ and a category with no requirement.

(f) Glass

Glass from windows can contaminate demolished material very easily. However, it is now unusual for glass to be present in large amounts in recycled aggregate as it is now standard practice to remove glass from buildings before demolition. Separation of glass from C&DW is a very difficult process as plate glass has a similar density to concrete or aggregate. The plentiful presence of glass in C&DW usually results in increased fines content following crushing. This is potentially harmful to concrete as plate glass could take part in alkali-silica reaction (ASR). In BS EN 12620-2:2002+A1:2008, glass was termed as Rg and recycled aggregates may be specified that have other constituents plus Rg content limited to 1% by mass.

3.3 PHYSICAL DIFFERENCES BETWEEN RECYCLED AND NATURAL AGGREGATES

It is believed that the impurities, particularly old cement paste stuck to RA have a significant influence on the strength of RAC. Several studies (Hansen 1985; Dae & Han 2002; Etxeberria *et al.* 2007) concluded that adhered mortar from the original

concrete plays an important role in determining performance with respect to permeability and strength.
As the adhered mortar is often prone to attract more water than the original aggregate, the absorption capacity of recycled aggregates is known to be greater than that of natural aggregate; this is believed to be one of the most significant factors distinguishing the two types of aggregate. The absorption capacity is related to the size of RA; (Hansen 1983) reported that the absorption capacity is about 3.7% for 4-8 mm RA, and about 8.7% for the 16-32 mm sizes, meanwhile it was only 0.8-3.7% for natural aggregates.

A concrete with recycled coarse aggregate and natural sand typically needs 5% more water than conventional concrete to match the workability of similar natural aggregate concrete (Buck 1977; Hansen 1983; Ravindrarajah & Tam 1985; Etxeberria *et al.* 2007). However, the workability of the recycled aggregate concrete is also influenced by its shape and texture (Shokry & Siman 1997).

The presence of adhered mortar on the surface of RA results in the formation of two interfacial zones (the 50 μm film between the coarse aggregates and the mortar) surrounding RA particles when used in RAC concrete, where the other interfacial zone is the old one previously formed between the adhered mortar and the aggregate particles. In contrast, in NAC concrete there is always just one interfacial zone. Several researchers have considered this as a major difference.

It was found (Hasaba *et al.* 1981) that the crushing of 24 N/mm^2 concrete results in around 35.5% of the old mortar attached to natural gravel of particle size of 5-25 mm: corresponding figures were 36.7% mortar for 41 N/mm^2 and 38.4% for 51 N/mm^2. Similar work (Hansen & Narud 1992) shows that the volume percentage of mortar attached to natural gravel particles was between 25% and 35% for coarse recycled aggregates of 16-32 mm size, around 40% for 8-16 mm size and around 60% for 4-8 mm. A recent study (Etxeberria *et al.* 2007) showed that crushed concrete comprises 49.1% of original aggregate plus the adhered mortar and 43% of original aggregate, 1.6% ceramic, 5.3% bitumen and 0.8% other materials. In this study, the total quantity of adhered mortar was estimated to be in the range 20-40% of the aggregate weight. This study also showed that the smaller the size of the aggregate the more adhered mortar. Therefore fine recycled aggregate would often contain more adhered mortar and have more absorption capacity. However, the utilisation of recycled fine aggregate for

RAC concrete is usually avoided due to its higher absorption capacity and increased shrinkage (Hansen 1996). The findings of the aforementioned studies are in agreement.

With respect to consistency**,** RAC concretes, in particular those containing more aggregate created from bricks, have been reported to be harsher and less workable than NAC concrete mixes, which is in most cases attributed to RAC's higher absorption rate compared to NAC. Many researchers (Buck 1997; Hansen 1983; Khalaf & DeVenny 2004; Khaldoun 2007; Etxeberria *et al.* 2007) have suggested pre-soaking recycled aggregates, in particular those derived from cement bricks, in water for some time so that recycled aggregates can be introduced to the mixer in a saturated surface dry condition (SSD); this will compensate for the loss in free water most likely absorbed when very dry aggregates are used.

Although the use of pre-soaked aggregate is possible at small scale *i.e.* laboratory conditions, from a practical point of view, it is seems to be difficult to keep aggregates in SSD condition. Extra water will result in increased w/c ratio and reduced strength. The use of workability agents such as an SP is a much better alternative (Poon 2004).

Bleeding (migration of mixing water to the top surface zone of the concrete section) due to use of RA is generally similar to that of natural aggregates. However, bleeding was observed to be reduced with recycled aggregates produced mainly from other materials rather than those originally produced from crushed concrete, such as the aggregates from cement, clay bricks, *etc.* (Khalaf & DeVenny 2004).

With regard to physico-mechanical properties, studies have shown the type of coarse aggregate (particularly its mineralogy) has significant influence on these properties (in addition to other factors). The importance of mineralogical characteristics of coarse aggregates on the quality of concrete has been pointed out by a number of researchers (Aïtcin & Mehta 1990; Baalbaki *et al.* 1991; Giaccio *et al.* 1992; Beshr *et al.* 2003). It was reported that different natural aggregates have yielded different concrete properties even with similar mix proportions under the same conditions. For instance, the effect of three different coarse aggregates used to produce concrete of low w/c ratio (0.24) was studied (Aïtcin & Mehta 1990); the results revealed that for a calcareous-lime stone aggregate (85% calcite), a dolomitic-limestone (80% dolomite) and a quartzitic-gravel aggregate (containing schist), the 91 day compressive strengths were 93, 103, and 83 N/mm^2 respectively. Moreover, the study concluded that the cement paste-aggregate bond was stronger in the limestone aggregate than the gravel concrete due to interfacial interlocking effects. Therefore, the potential to develop different concrete properties is most likely high for RAC concrete; quality of recycled aggregates is more variable than natural aggregates even for aggregates produced from the same source of C&DW. When RAC concrete is made with 100% RA, the compressive strength is lower than that of conventional concrete of similar w/c ratio, but larger for RAC with lower w/c ratio (Tavakoli & Soroushian 1996). For RAC concrete prepared with 100% coarse RA, it would be necessary to use about 20 to 30% extra cement to achieve a compressive strength similar to that of natural aggregate concrete, and therefore this is an uneconomic approach (Etxeberria *et al.* 2007). Previous studies indicate that, compared with NAC concrete, the RAC concrete could still achieve at least two-thirds of the required compressive strength and modulus of elasticity, with satisfactory workability and durability (Vivian & Tam 2007).

3.5 CLASSIFICATION OF RECYCLED AGGREGATES

There have been several attempts introduced in the past to classify recycled aggregates. This section will present some the most recognised ones. BS 6543:1985 permitted the use of by-products and waste materials in building and civil engineering. However, this standard contains guidance concerning the use of recycled C&DW as aggregate in new concrete; it was stated that clean aggregates can be used to produce concrete of low strength but no other information was given concerning its production or performance.

(Mulheron 1988) split the available recycled material into four main categories:

1) Crushed demolition debris (mixed crushed concrete and brick that has been screened and sorted to remove excessive contamination).
2) Clean graded mixed debris (crushed and graded concrete and brick with little or no contamination).
3) Clean graded brick (crushed and graded brick containing less than 5% concrete or stone and little or no contaminant).
4) Clean graded concrete (crushed and graded concrete containing less than 5% brick or stone and little or no contaminant).

There is a large variation in material produced from different recycling operations due to the lack of a well-defined and unified recycling process. Operators did not have standards to follow to produce acceptable recycled products; this study recommended that until this situation changed, consumer confidence would remain low. Classification of recycled products under these conditions seems unlikely to be possible. Another notable classification (RILEM 1994) was that based on the maximum allowable values for impurities in recycled aggregates. These values are re-introduced here as displayed in Table 3.2.

Table 3.2 Classification of recycled coarse aggregates for concrete (RILEM 1994)

Mandatory requirements	Type of aggregate		
	1	2	3
Minimum dry particle density (kg/m^3)	1500	2000	2400
Maximum water absorption (%)	20	10	3
Maximum content of material with SSD < 2200 kg/m^3 (%)	-	10	10
Maximum content of material with SSD < 1800 kg/m^3 (%)	10	1	1
Maximum content of material with SSD < 1000 kg/m^3 (%)	1	0.5	0.5
Maximum content of foreign materials (glass, bitumen, soft materials, *etc.*)	5	1	1
Maximum content of metals (%)	1	1	1
Maximum content of organic material (%)	1	0.5	0.5
Maximum content of filler (< 0.063 mm) (%)	3	2	2
Maximum content of sand (< 4 mm) (%)	5	5	5
Maximum content of sulphate (%)	1	1	1

SSD is the saturated surface dry density.
Type 1 aggregate is composed of 100% recycled brick.
Type 2 is 100% recycled concrete.
Type 3 is a blend of natural and recycled aggregates.

According to RILEM Technical Committee 121-DRG (RILEM 1994), recycled aggregates were classified into three groups: Group I: aggregates produced mainly from masonry rubble, Group II: aggregates obtained from concrete rubble, Group III: a mixture of NAs (> 80%) and rubble from the other two groups (with up to 10% of Group I, and Group II aggregate). These can be used for the production of all types of concrete. It was recommended that up to 20% of coarse NA can be replaced by RA for the proportion of new concrete.

Recycled aggregates from old roads are usually uniform, but this is not always the case for C&DW; rubble is collected from various sources and the properties of the aggregate are unlikely to be consistent. This leads to difficulties in the application of the resulting aggregates for the production of new concrete (BRE 1993). The cleaner the aggregate, the stronger the concrete (Amnon 2004).

In BS 8500-2:2002, which complements BS EN 206-1-2000, recycled aggregates are subdivided into two separate classes: Class I: RCA created predominantly from crushed concrete which is specified for use in concrete up to strength class C40/50 (40 and 50

N/mm^2 are the minimum characteristic cylinder and cube compressive strength respectively) and durability classes X0, XC1-4, DC1, and XF1, and Class II: RA produced from processed C&DW which is only permitted provided that additional provisions are included in the project's specifications to account for potential variability of the RA. Provisions are based on the composition of the proposed RA.

Recently, a study was undertaken by Wrap (www.wrap.org.uk) to investigate the possibility of using an alternative method for classifying recycled aggregates. The aim was to overcome current barriers and concerns that restrict their specification and use in concrete. Three classes were suggested: Class A: RA that may be used in a wide range of concrete including marine environments, Class B: RA used in a combination of natural and recycled aggregate that may be used for moderate exposure conditions, and Class C: RA used for concrete subject to only the mildest exposure conditions. More recently, BS EN 12620:2002+A1:2008, classifies recycled aggregates into different categories as specified in Table 3.3.

Table 3.3 Classification of the constituents of coarse recycled aggregates (BS EN 12620:2002+A1:2008)

Constituents	Content percentage (by mass)	Category
RC (Concrete, concrete products, mortar, concrete masonry units)	≥ 90	Rc_{90}
	≥ 80	Rc_{80}
	≥ 70	Rc_{70}
	≥ 50	Rc_{50}
	< 50	$Rc_{Declared}$
	No requirement	Rc_{NR}
Rc+Ru (Unbound aggregate, natural stone , hydraulically bound aggregate)	≥ 95	Rcu_{95}
	≥ 90	Rcu_{90}
	≥ 70	Rcu_{70}
	≥ 50	Rcu_{50}
	< 50	$Rcu_{Declared}$
	No requirement	Rcu_{NR}
Rb (Clay masonry units *i.e.* bricks and tiles, calcium silicate masonry unit, aerated non-floating concrete)	≤ 10	Rb_{10}
	≤ 30	Rb_{30}
	≤ 50	Rb_{50}
	> 50	$Rb_{Declared}$
	No requirement	Rb_{NR}
Ra (Bituminous materials)	≤ 1	Ra_{1}
	≤ 5	Ra_{5}
	≤ 10	Ra_{10}
X+Rg X: others such as cohesive (*i.e.* clay and soils). Miscellaneous: metals (ferrous and non-ferrous), non-floating wood, plastic and rubber, gypsum plaster. Rg: glass	≤ 0.5	$XRg_{0.5}$
	≤ 1	XRg_{1}
	≤ 2	XRg_{2}
FL	≤ 0.2 [a] (cm^3/kg)	$FL_{0.2}$
	≤ 2	FL_{2}
	≤ 5	FL_{5}

[a] The ≤ 0.2 category is intended for special applications requiring high quality surface finish.

In a study (Vivian & Tam 2007) a classification system for RA was proposed; the system classifies RA into grades depending on some properties as shown in Table 3.4.

Table 3.4 Classification of RA (Vivian & Tam 2007)

Properties	Grades						
	A	B	C	D	E	F	G
Particle density (kg/m^3)	> 2.5	2.49-2.4	2.39-2.3	2.29-2.20	2.19-2.10	2.09-2.00	< 2.00
Water absorption (%)	< 1.0	1.1-3.0	3.0-5.0	5.1-7.0	7.1-9.0	9.1-10.0	> 10.0
Flakiness index (%)	< 8	9-16	17-22	23-28	29-34	35-40	> 40
Ten percent fines value (kN)	> 150	149-120	119-110	109-100	99-80	79-50	<50
Aggregate impact value (%)	< 20	21-23	24-26	27-28	29-31	32-35	> 35
Chloride content (%)	< 0.015	0.016-0.03	0.031-0.05	0.051-0.100	0.101-0.500	0.501-1.00	> 1.00
Sulphate content (%)	< 0.015	0.016-0.03	0.031-0.05	0.051-0.100	0.101-0.500	0.501-1.00	> 1.00

Grade A represents the highest quality, and grade G the lowest.

3.6 BARRIERS TO RECYCLING

The Department of the Environment issued Mineral Planning Guideline 6 in 1994, which committed the construction industry in the UK to a 100% increase in the use of secondary and recycled materials by 2006. The guideline suggests that this increase can be achieved by using recycled aggregates wherever they can replace natural aggregates technically, economically, and environmentally. However, it was stated (Carpenter 1994) that there is a price to pay for being eco-friendly and that potential uses of secondary aggregates are hindered by consumer tastes and over conservative construction specifications. At present the cost of primary aggregates is still similar to that of recycled aggregates; therefore, material suppliers, contractors and the construction community as a whole are not likely to accept the use of recycled aggregates in construction with their variability in composition and properties and lower performance, when an economical proven alternative is available.

(Watson 1993) stated that the UK government needed to take determined action if the use of recycled aggregates is to be encouraged; a number of options were available to the government: (a) The price of natural aggregates could be increased by more stringent controls on their extraction or by placing a levy on primary aggregates. (b) A levy on mineral waste dumping could have been introduced (c) A change could have been made to road specifications to accommodate the use of recycled aggregates in place of primary aggregates. (d) The development plan policies needed to be drawn up to save high-grade materials for high-grade applications. All this would have cleared up a lot of uncertainty surrounding these materials and could result in more acceptance of recycled materials with confidence.

(McLaughlin 1993) cited over-specification as the major factor behind the lack of use of recycled aggregates when he reported that designers were adopting a conservative approach as a result of concern regarding risk and liability. This study was in agreement with (Rockliff 1996); the author believed that good quality aggregates were unnecessarily being used in non-structural fill layers when instead they should have been used in asphalt or concrete. So if more use of recycled materials is to be made, then responsibility lies with clients, funding institutions, and designers, who should incorporate sustainability into the design (Walsh 1997). There is also a complete lack of standards for the use and production of recycled materials in construction; this simply meant that there was no motivation for demolition contractors to improve the quality of their product. A study (Chevin 1990) reported that many highway specifications set by highways agencies were written around trusted recipes which could inhibit the scope for recycling in some instances. Those involved in specifying materials for construction projects have a preference to use materials with a proven performance history, and as a result many consultants in the UK will not risk specifying recycled materials. This reluctance to use secondary materials is often the result of a lack of well documented laboratory tests and field trials.

One of the main difficulties with recycling C&DW materials is that demolition works are not designed for good materials recovery, thus producing mixed wastes which are not suitable for utilisation (Aggregates Advisory Service 1999). Many case studies have shown that with proper planning before demolition starts, materials can be recycled more easily and more economically. In addition, a lot can be retrieved and learnt from these cases; the demolition of structures should indeed be considered during the planning stages before construction to ensure that relatively little waste is produced. Technical aspects of recycling should be considered during design, construction, and maintenance. This involves designing elements that are simple to dismantle, limiting the use of bonded materials. A structure should be designed with several use options in addition to its initial use.

The UK's Department of the Environment, in 1997 suggested policy measures which should be undertaken to encourage the reuse and recycling of waste products (DeVenny 1999). These were:

- Increasing the cost of waste disposal.
- Increasing the price of primary aggregates.
- Developing specifications that do not exclude the use of recycled materials as aggregates.
- Funding research into the performance of recycled materials and publishing the findings.
- Providing information on opportunities for using alternative material as aggregates.
- Identifying suitable sites for recycling plants and good management practice.

Direct participation by government agencies was seen as the best way to lead by example and these opportunities included:

- Encouraging and accepting the use of recycled products in contract documents as alternatives to primary products.
- Setting a minimal requirement of recycled products (dependent on structure type) for public construction projects.
- Undertaking pilot projects with maximum use of recycled products and then publicising the results.

The British Government is committed to sustainability, for instance, the Governmental Panel on Sustainable Development confirmed in 1998 that "the use of recycled materials in all aspects of building will make an important contribution to reducing the sector's impact on the environment", but the industry in general feels that until other government departments work in harmony, progress in the use of recycled materials will be slow (Smith 1988). In 2008, the Scottish Government has proposed raising Scotland's national recycling rates with the aim of rising from the current 30% rate to a 40% recycling level in 2010 and then up to 70% by 2025. Germany, UK and the Netherland are cited by many authors as the most advanced recyclers of C&DW.

3.7 ECONOMICS OF RECYCLING

All companies requiring coarse aggregate for use in their operations are faced with an economic choice. When recycled aggregate is considered, they need to decide if using natural aggregates will be more cost effective than using recycled aggregate. Factors such as client opinion, transportation costs, taxes, quality of materials, their performance, the availability of landfill sites as well as the concrete producer experience will be considered in this decision.

The Government endeavours to make recycling of C&DW the most economical option. Due to landfill taxes it is relatively cheaper to take C&DW to a recycling site as the cost of transport is the only cost. This cost would still be present even when transporting the material to a landfill site. The current landfill tax is £2 *per* tonne plus a gate charge at landfill sites. Recycled aggregates cost relatively less *per* tonne than aggregate extracted from primary resources especially since the Government imposed an aggregate levy of £1.60 *per* tonne on natural aggregates.

The segregation of materials at the demolition source is also an economic factor. As demolition involves the removal of a structure, which is no longer required, there is an incentive to get rid of it as quickly and cost-effectively as possible.

There are also hidden economic costs, which need to be considered when choosing between recycled and natural aggregates. The most important is the environmental cost of obtaining natural aggregates from their source, as they can never be replaced. Other costs include visual disturbance left from the extraction process, which will also create noise, dust, and vibration. However for the production of recycled aggregates, these factors will also be relevant. Transport costs will also be an economic factor for both types of aggregate. There is a potential further cost arising from using additives, particularly with recycled aggregates, which should also be taken into account

It was reported (Webb 1999) that there was a great deal of debate going on between the British Aggregate Construction Materials Industries (BACMI), the representative of the aggregate suppliers, and the then Department of the Environment. The Government wanted to put a levy on the use of primary aggregates to stimulate the use of recycled aggregates and meet forecaste aggregate demand of 421-490 million tonnes *per* year by 2011. Statistics at that time (1991) showed that only about 10% of the aggregates used were derived from recycled sources. There was disagreement; the BACMI and the Sand and Gravel Association (SAGA) felt that this was unfair and that the government was restricting the trade of the companies it represented. In 1996 the Landfill Tax Regulations were introduced as a measure to minimize the amount of waste going to landfill. Since 2006, all waste has been taxed at £21 *per* tonne with lower risk waste such as clean building rubble staying at £2 a tonne. The £2/t figure is set to rise by £3 *per* tonne *per* annum until it reaches £35.

3.8 PREVIOUS RESEARCH ON THE PROPERTIES OF RAC

It is believed that impurities, particularly old cement paste clinging to RA have a significant influence on the strength of RAC. The author has reviewed many studies which concluded that adhered mortar from the original concrete plays an important role in determining the performance with respect to permeability and strength. This was considered to be one of the most significant differences between RAC concrete and NAC concrete (Wrap 2007).

According to (Wrap 2007), previous research on RAC can be split into two groups: i) RAC with RA containing 5% masonry (the rest are aggregates of different origin), and ii) RAC with RA that can contain up to 100% masonry.

Much research has been previously carried out to investigate the possibility of replacing natural aggregate with its recycled equivalent, in particular coarse aggregate, in new concrete mixes at different percentages. The effects of replacement on the behaviour and performance of the RAC were studied. The substitution of natural fine aggregate with recycled fine materials is usually avoided; the main reason for this situation is that the latter typically exhibits high absorption and shrinkage that leads to a weak performance of the matrix both in mechanical and durability-related terms, mostly for substitution levels of more than 30% (Wainwright 1993).

The following parts of the literature review will focus on the characteristics and performance of RAC made with RA, derived predominantly from crushed concrete as only this type will be used for the experimental programme (Chapters 5 and 6). This part is structured to review investigations that focused on testing of physico-mechanical properties of the RA and RAC, durability related issues and to a lesser extent the mix proportioning, and the RAC's stress-strain behaviour.

(a) Workability, strength and durability of RAC

It has been reported (Bairagi & Kishore 1993) that earlier studies indicated that little change in workability of RAC was noticed; usually 8-10% more water was required, but the loss of workability with time is faster. In this study, RA was obtained by processing natural concrete produced in the laboratory by jaw crusher and used to replace the crushed black basalt stone in RAC mixes. RA replaced NA at levels of 25, 50, 75 and

100%; a maximum strength reduction of 15% was observed when up to 50% of NA was replaced by RA. (Bairagi & Kishore 1993) also reported previous studies citing approximately 10% lower compressive strength, 0-20% reduction in tensile and flexural strength, and 10-40% lower modulus of elasticity. Creep and shrinkage were increased by 30-50% and 20-70% respectively.

(Dhir 1998) contends that the use of up to 30% RA to replace NA will not have significant adverse effects on RAC cube strength. For higher RA contents, minor alterations to the mix proportions may be needed to ensure that equivalent performance to NAC is achieved.

In a study by (Niro *et al* 1998), the recycled aggregate was obtained from demolished concrete structures, processed in a laboratory, in which impurities such as wood, glass, plastics, wires, and bricks were removed. The aggregate was then used to replace natural aggregate in RAC concrete at different levels of replacement (0, 7, 9, and 20%), for two different grades of concrete (30 and 40 N/mm^2). The study showed that it was impossible to make RAC concrete stronger than 35 N/mm^2 when $0.45 \leq w/c \leq 0.50$, and even with w/c 0.35 the natural aggregate concrete achieved 60 N/mm^2, while for similar mix proportions RAC has produced only 35 N/mm^2 cube strength. Blending of natural and recycled aggregates did not result in significantly improved cube strength at high w/c ratio; the greatest improvement was less than 10%. Tensile strengths and elasticity modulus were found to follow the same trend as the compressive strength while the workability was little improved. In contrast, (Fong & Jaime 2002) reported that slump loss of concrete will be quite fast for RAC without pre-wetting of RA.

(Khaldoun 2007) carried out an experimental programme to study the mechanical properties of RAC as compared with conventional NAC. RA created from laboratory-produced cubes crushed after 28 days with a maximum aggregate size of 19 mm, natural fine aggregate and OPC were used. Five different mix proportions with target cube strengths ranging from 20 to 50 N/mm^2 were produced. Because of the large absorption capacity of the RA, aggregates were maintained in a saturated surface dry condition before mixing and different percentages of SP used to adjust the slump to about 50-60 mm. Results showed that the compressive strength of RAC was about 90% of that of RAC and NAC exhibited similar behaviour with respect to the rate of strength development and workability.

Many researchers (Topcu & Guncan 1995; Van 1996; Mansur 1999; Amnon 2003 & 2004; How 2003; Khalaf & DeVenny 2004; Jianzhuang *et al.* 2005) have reported that compressive strength of RAC was lower than that of NAC. It was reported (Sagoe 2001) that the basic characteristics of RA, such as, gradation, specific gravity, absorption capacity, dry density, soundness, and resistance to wear were generally worse than those of NA mainly because of the presence of residual mortar and impurities. Workability is usually affected by the aforementioned properties. Due to the large amount of old mortar which is attached to the recycled aggregates, the modulus of elasticity of recycled aggregate concretes is always lower than that of corresponding control concrete, particularly when recycled sand was used, however, comparatively high elastic moduli are reported for recycled aggregate concretes produced with coarse recycled aggregate and natural sand (RILEM 1992).

It was reported (Niro *et al.* 1998) that at ordinary mixes with usual w/c ratio (> 0.5) and even with w/c of 0.35, RAC strengths are far below similar NAC concrete; for instance, the compressive strength of an NAC concrete mix was 60 N/mm^2, while similar RAC concrete strength was only 35 N/mm^2. Therefore it was concluded that to attain a satisfactory strength of RAC concrete it is necessary to use mixes in which RA and NA are blended in different proportions and to use high strength cement.

(Amnon 2003) used concrete cubes crushed at different ages as a source of RA for concrete with up to 100% replacement to investigate the properties of RAC. Results of this extensive study showed the quality of RAC is less than NAC; the average compressive strength loss was 24%. Other properties, such as flexural and splitting strengths, absorption and absorption rate, drying shrinkage and depth of carbonation, exhibited similar trends. The ratio of the flexural and the splitting strengths to the compressive strength was in the range of 16-23% and 9-13% respectively; these values are about 10-15% lower compared to the recommendations of ACI 363R. Workability and cohesion of RAC were slightly affected by the insufficient amount of fines in the RA, but water requirement for a given slump was not significantly affected by the type or age of the aggregate.

For concrete made with 100% RA, the compressive strength and modulus of elasticity of RAC was reportedly decreased at different percentages. For instance, (Bairagi & Kishore 1993) reported 40% reduction in compressive strength, Yamato (1998) measured a 45% decrease, (Ravindrarjah 2000) measured a 9% decrease and (Amnon 2003) reported 25% reduction.

In a similar trend a reduction in the modulus of elasticity was measured for the 100% RA concrete, for instance (Frondistou 1977) reported up to 40% decrease at relatively higher water to cement ratio of 0.75 and insignificant decrease at a lower ratio of 0.55, (Andreas & Rühl 1989) reported a decrease of 12%, (Gerardu & Hendriks 1985) estimated 15% decrease and (Jianzhuang *et al.* 2005) reported 45% reduction. Recently (Etxeberria *et al.* 2007) reported that for full replacement, the compressive strength was 20 to 25% less than similar conventional concrete, and the reduction of elastic modulus (16%) was dependent on the level of RA replacement. The given examples show notable variation of strengths and stiffness of RAC; this however could be attributed to the inconsistency of properties of the recycled aggregates used by various investigators.

Another study (Salmon & Paulo 2004) in which fine and coarse aggregate produced from old bricks and concrete rubble was used at different percentages, concluded that natural aggregates can be replaced by 20% recycled aggregate from old concrete or masonry, the resulting RAC will likely present similar, and sometimes better, behaviour than the reference concrete made with natural aggregates However, this conclusion is in agreement with the RILEM Technical Committee report- RILEM 121-DRG (RILEM 1993). Recent research (Etxeberria *et al.* 2007) conformed with these studies; it showed that RAC made with 25% recycled coarse aggregate achieved the same mechanical properties as a similar conventional concrete, while RAC made with 50-100% of recycled coarse aggregate needs 4-10% more cement plus 5-10% less effective w/c ratio.

In contrast to compressive strength and modulus of elasticity, several researchers (Tavakoli & Soroushian 1996; Sague *et al.* 2001; Etxeberria *et al.* 2007) have reported that the tensile strength of RAC is similar or usually better than that of NAC concrete. The difference in the tensile strength of RAC and reference concrete at 28 days was less than 10% (Ajdukiewicz & Kilszczewicz 2002).

A recent study led by Dhir was carried out at the Concrete Technology unit of Dundee University, UK, for the Waste Resources Action Programme (Wrap 2007), UK. It was reported that a total of 125 concrete mixes were cast and tested using a number of different aggregates: natural aggregate, three laboratory-produced crushed concretes, three types of crushed bricks (crushed in the laboratory), and eight combinations of brick and crushed concrete aggregates. Tests were carried out for: cube strength, flexural strength, elastic modulus, drying shrinkage, initial surface absorption, carbonation resistance, chloride ingress, freeze/thaw, abrasion, sulphate attack, and leaching. Results showed that the use of RA at 20% by mass of aggregate had little effect on the performance of RAC (RA in this study refers to recycled aggregates that may contain up to 100% brick aggregate). It was also concluded that the proportion of brick within RA when used at moderate levels was not significant.

Other pozzolanic materials were also examined to produce RAC concrete, although these are in short supply compared to fly ashes. For instance, the most recent study (Gonzalez & Martinez 2008) incorporated RA and silica fume. RA in this study was obtained from a mobile plant that crushes only clean concrete (free of impurities); RA was washed and principally formed of 72% concrete and 20% stone. The workability in this study is guaranteed by using a water-reducer at different levels to ensure the design slump (50-100 mm). Results revealed that it was possible to produce RAC concrete that achieved similar compressive strength as the NAC (30 N/mm^2 at 28 days) using 50% RA (and 50% NA) by the addition of 6.2% cement. RAC with the addition of silica fume (8% of the cement by mass was added without addition or reduction of cement) produced similar strength to the control mix. Results also showed that silica fume did not modify the tensile strength and modulus of elasticity.

However, several studies showed that concrete mixes incorporating silica fume were tending to require higher dosages of superplasticizer, for instance (Mazloom *et al.* 2004) stated that high dosages of SP (a water-reducer or combination of water-reducer and a retarder) is required to enhance the workability of concrete made with silica fume. That is predominantly due to its irregular shape and the smaller size of particles (0.1-0.2 µm). Another example, (Hassan 2000) reported that a high SP dosage (31 kg *per* 100 kg of binder) was required to keep the workability of concrete containing silica fume and fly ash. Silica fume is also an expensive admixture compared to other mineral admixtures; this could have an obstructive impact on the overall cost of concrete.

(b) Stress-strain relationship of RAC

The compressive strength is the peak value of the stress–strain curve, and the area under the ascending portion of the curve provides a measure of the toughness of concrete, meanwhile the descending portion of the curve indicates its ductility. The descending part is essential when the concrete structures could be subjected to dynamic loading such as impact, earthquakes, or fatigue loading (Jianzhuang *et al.* 2005).

A study (Bairagi & Kishore 1993) revealed that the replacement ratio [RA/(NA+RA)] has a marked influence on stress-strain relations, particularly at higher ratios, it was found that for concrete containing higher proportions of RA, strain increases at a faster rate than the applied stress.

Another investigation carried out by (Jianzhuang *et al.* 2005) focused on failure behaviour and the influences of the RA content on compressive strength, elastic modulus, peak and ultimate strains of RAC. This study revealed that the stress–strain relationship for RAC was quite similar to that of NAC.

(Topcu & Guncan 1995) produces complete stress–strain curves for RAC with replacement percentages of 0, 30, 50, 70 and 100%. It was concluded that with the increased amount of RA, values of compressive strength, toughness, elastic energy, plastic energy and the elastic modulus were decreased. Research indicates that the elastic modulus of RAC is about 30% less than NAC concrete attaining equal compressive strength (Katharina 1997). An investigation (Jianzhuang *et al.* 2005) examined the influences of RA content on the compressive strength and the elasticity modulus of RAC. RAC mixes with different RA replacements at levels of 30, 50, 70 and 100% were studied. A typical set of stress-strain curves for the RAC at different RA replacements are displayed in Fig. 3.1.

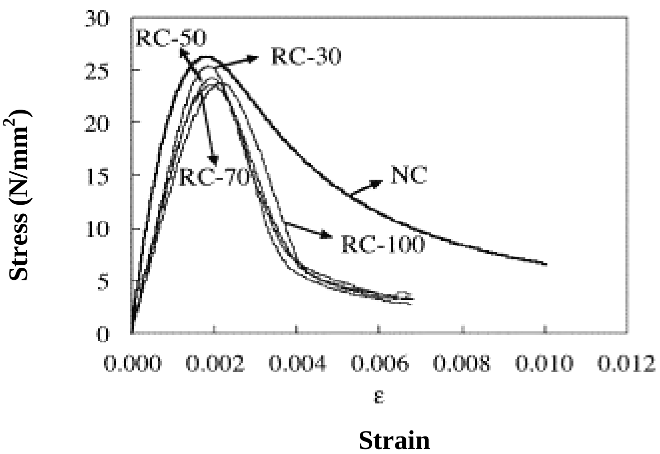


Fig. 3.1 Typical stress-strain (σ-ϵ) curves of RAC (Jianzhuang *et al.* 2005)

(Note: in Fig. 3.1, RC-Recycled Aggregate Concrete, NC-Natural Aggregate Concrete, RC-30 means 30% RA replacing NA).

It was concluded that the substitution of RA leads to a substantial change in stress–strain responses. This change is generally characterized by an increase in the peak strain (strain at peak stress), and a significant decrease in the ductility of the concrete as described by the descending portion of the stress-strain curve. The peak strain of RAC is lower than that of NAC; it was increased by the increase of RA; for a 100% replacement the peak strain was increased by 20%. Fig. 3.1 shows that the slope of the descending branch of the curves is decreasing as the RA content increases. It also can be seen that while strain at peak stress can be estimated as 0.0019 for NAC, it is 0.0022 on average for RAC. Thus, RA, used in this study, was slightly more deformable than NA.

It was reported (Khaldoun 2007) that strains at peak compressive stress in RAC were 5.5% larger than in NAC, but would not have significant implications for structural design. Shear strength of concrete depends significantly on the ability of the coarse aggregate to resist shearing stresses. RA used is relatively weaker than NA in most cases and yielded reduced shear strength. Experimental data has shown that for NAC, shear cracks propagate in the hardened cement matrix and around the relatively stronger coarse aggregate (Angelakos *et al.* 2001). For HSC where the matrix is relatively stronger, shear cracks pass through the matrix as well as the aggregate, forming a smoother crack surface. The modulus of elasticity for RAC has been reported to be in the range of 50-70% of the normal concrete (Akash *et al.* 2007).

However, in general most of the studies reviewed have reported that the shape of the stress-strain relationship of RAC is very similar to counterpart NACs. Therefore, RAC structures can be theoretically designed just like NAC structures, however for a particular case when strain at peak stress is shown to be excessive; an extra partial safety factor may need to be incorporated into the design process when RAC is going to be used in structural concrete.

Very limited data are available on many concrete aspects such as abrasion resistance, bond strength, creep and shrinkage and most durability-related characteristics. With respect to abrasion (Gilpin *et al.* 2004; Khalaf & DeVenny 2004) reported that recycled aggregates are extensively used in USA, UK and other countries as new material for rigid pavements. For shrinkage, (Amnon 2003) reported that the shrinkage could be about 0.55-0.8 mm/m, whereas the comparable value for NAC is only 0.30 mm/m. The use of RA in concrete includes a large shrinkage due to the high absorption (Akash *et al.* 2007). With respect to creep, (Ajdukiewicz & Kilszczewicz 2002) states that some studies have shown that creep of RAC after one year, in a reverse tendency, was about 20% lower than similar NAC. A reduction of up to 10% in the bond strength of the RAC has been reported at 100% RA replacement (Ajdukiewicz & Kilszczewicz 2002). Durability properties can be improved by the use of supplementary cementitious admixtures such as fly ash, condensed silica fume, *etc*. (Ajdukiewicz & Kilszczewicz 2002; Akash *et al.* 2007). Concretes made with RAC using RA derived from air-entrained concrete were highly frost resistance, and the carbonation depth of RAC was found to be 1.3-2.5 times greater than that of the reference concrete (Akash *et al.* 2007).

It is quite noticeable that for most of the cases reviewed, research involving RAC was carried out using recycled aggregates mainly produced from crushing of laboratory prepared concretes smashed 28 days after casting. This is also the case for many other studies in which recycled aggregates were derived predominantly from concrete bricks, tiles and masonry or a combination of these materials (this research was all reviewed but not reported here as this study focused on RA from concrete).

Although limited data are available on commercial recycled aggregates, it is believed that the behaviour and properties of RAC in which recycled aggregates were derived from processed C&DW could be different. Both fresh and hardened properties may be affected. In a study by (Fong & Jaime 2002), it was observed that the nature of recycled aggregates from a recycling facility was found to be far more consistent than those from trial aggregates produced from C&DW collected from demolished concrete rubble. However, with respect to strength, the earlier studies showed that the strength of the source concrete has little effect on the performance of RAC (Khaldoun 2007; Wrap 2007).

3.9 UTILISATION OF PFA IN CONCRETE

(a) General

(King 2005) quoted Abdun-Nur who said ‘concrete which does not contain fly ash, belongs in a museum’. In modern concrete technology, the additions of active mineral additives such as fly ash, silica fume, slag, natural pozzolan, *etc.* is a measure of great technological and economical significance (Liu *et al.* 2000). However, published literature in the area in which SP and/or PFA were used to make RAC did not show much research, perhaps due to the limited use of RAC in the past. In contrast, data are available when natural aggregates were used; therefore, the review of previous work with NAC will help to understand the behaviour of these materials when used for RAC mixes.

BS EN 450-1:2005+A1:2007 defines fly ash as a fine powder of mainly spherical shape, having glassy particles, derived from burning of pulverised coal, with or without combustion materials, which has pozzolanic properties and consists essentially of silicon dioxide (SiO_2) and aluminium oxide (Al_2O_3).

Increasing urbanisation and improvements in both developed and developing countries will lead to more industrialization and mineral by-product wastes such as fly ash. Some of these by-products can be used in new concrete. The 21st century will become the golden age of environmentally friendly supplementary cementing materials for concrete; the use of such material as fly ash, blastfurnace slag, silica fume and condensed cements will lead to significant improvements in durability and structures with longer life and lower maintenance costs (A2E03 2006).

Most of the reasons for use of PFA in concrete as will be described in the paragraphs to follow are practical, such as improving workability, increasing strength and durability, decreasing heat of hydration; mainly in cement-rich mixtures, and decreasing permeability. In particular, high volume fly ash content (50-100% of the total cementitious materials) is considered to result in beneficial economic, health and environmental impacts; (Bouzoubaâ & Fournier 2003) reported that it is possible to replace 50% of cement by a fine fly ash (specific surface area of 3000 cm^2/g) that yielded a concrete with compressive strength similar to the 28 day strength of the control concrete, and results in a cost reduction of about 20%. The use of fly ash is accepted in recent years primarily because of economy from saving cement, secondly because of consuming industrial waste and thirdly because of making durable concrete (Liu *et al.* 2000).

Pulverised fuel ash (PFA) is used nowadays in producing ready-mixed cement. In this type of cement, PFA is mixed in a dry base with cement in different percentages (mostly below 30%) and packed for general construction applications. The quality of these cements is no different from ordinary Portland cements (Massazza 1993). Pozzolanic cements showed: lower early strength, higher later ultimate strength, lower heat of hydration and lower permeability.

Fly ashes have a successful history of use in concrete all over the world for over 70 years. For instance, in the UK, as a result of early research work carried out by Fulton and Marshall (Advanced Concrete Material 1990), the Lednock, Clatworthy and Lubreoch dams were constructed during the 1950s with fly ash as a partial cementitious material. These structures are still in excellent condition, after some 50 years of service. Thomas reported that cores taken from 30 year old fly ash concrete structures in general have performed well although concretes have been made with unclassified fly ashes over which minimal control of fineness, loss on ignition (LOI), *etc*. have been exercised. Recently, some of the most important projects worldwide have relied on PFA concrete. The list covers but is not limited to precast and on-site applications including dams, power stations, offshore decks and platforms, residential and office buildings, bridges, silos and highways. It is outwith the scope of this study to count and to prepare a list of the projects in which PFA was employed for different purposes; instead a few examples will be presented here. For instance in the UK, it was used in the construction of first commercial pressurised light water reactor power station, Sizewell B, Suffolk (Davies & Kitchener 1996). In the USA, PFA was used in an architecturally exposed concrete six storey building for the Jonas Salk Institute of Biology, San Diego, California. In Germany, the Castor and Pollux towers in Frankfurt, and the cooling tower of new 950 MW power stations at Niedraubem; the design requirement was to increase the floor space, therefore high strength concrete (grade B65) was used containing 400 kg cement and 100 kg fly ash /m^3 for the inner walls. The columns were constructed from high strength concrete (grade B115) with 470 kg cement and 120 kg fly ash and 35 kg micro-silica *per* m^3 (silica fume) the strength of the columns reached 130 N/mm^2 at 90 days (ECOBA 2006). The Parklane Development in Halifax, Nova Scotia, Canada is a seven storey structure and was built with 55% high-volume PFA concrete (Wilbert 1998). In China, fly ash was extensively employed for the Three Gorges Dam Project; this dam is currently the biggest hydropower-complex in the world. About 30% fly ash of grade I (according to China National Standard GB 1596-91) was used to replace cement in the considerable volume of concreting works (Gao *et al.* 2007).

High volume fly ash was mostly used in mass concrete, *e.g.* roller-compacted dams and highway base courses where high strength and high workability were not required (Poon & Wong 2000). High volume fly ash concrete for structural use was developed by the Canada Centre for Mineral and Energy Technology (CANMET) in 1985. This type of concrete has typically 50–60% fly ash as the total cementitious material content. Superplasticizers (high-range water reducing admixtures) are used to keep good workability. Successful applications of this type of concrete, including concrete columns, with a compressive strength requirement of 50 N/mm^2 at 120 days, and piles with the compressive strength requirement of 45 N/mm^2 at 28 days (Bilodeau *et al.* 1993; Poon & Wong 2000) are notable.

Previously, the use of fly ashes in concrete has been permitted by many codes worldwide. For instance, in the UK, British Standards dealing with fly ashes were published for different purpose, these are: BS 3692:1965 (in which fly ash was considered as fine aggregate), BS 3692-1:1982 (fly ash for use as a binder in concrete), BS 3692:1984 (fly ash for miscellaneous applications), BS 3692-1:1993 to replace the 1982 version, BS EN 450:1995 European Standard for fly ash, BS 3692-2:1996 for use of fly ash as a Type I addition, BS 3692-1:1997 to replace the 1993 version, BS 3692-2:1997 for use of fly ash in cementitious grouts, BS EN 451-2:1995 for determination of fineness by wet sieving, and BS EN 451-1:2003 for determination of free calcium. BS EN 450-1 and 2 were published in 2005 for use of fly ash in concrete and finally BS EN 450-1:2005+A1 2007 superseded BS EN 450-1:2005 and BS 3692-1:1997. Fly ash for use as a mineral admixture in concrete is also covered in a specification by the American Society for Testing and Materials (ASTM) which is widely used in North America and many places around the world. To make a comparison between some properties Table 3.5 is prepared as follows.

Table 3.5 Comparison of standards for fly ash

Property	BS 3892-1:1997	BS EN 450:1995	BS EN 450-1:2005+A1 2007
Fineness, max. retained on 450 μm (%)	12.0	40.0	40.0 for category S, and 12.0 for category N
Fineness variation	± 10 on average value	± 10 on average value	± 10 on average value
Loss on Ignition (LOI), max. (%)	7.0	5.0	Up to 9.0
Relative density (kg/m^3)	2000	± 150 on nominal value	± 200 on nominal value
Free calcium oxide (%)	-	2.5	2.5 for CaO-free, and not more than 10% for reactive CaO
Magnesium oxide (MgO), (%)	-	-	4.0
Sulfuric anhydride (SO_3), (%)	2.0	3.0	3.0
Total of the three oxides above	10	10	-
Reactive silicon dioxide (SiO_2)	-	25	25
Chloride, max. (%)	0.1	0.1	0.1
Moisture content, max. (%)	0.5	dry	-
Water requirement, max. (%)	95	-	For category S 95, but not applied for category N
Activity index, min. (%)	80 (28 days)	75 (28 days), 85 (90 days)	75 (28 days), 85 (90 days)
Soundness (mm)	10	10	10

Activity index is the ratio of the strength of cement paste prism to the strength of binder (cement +PFA) for the same w/c (0.5).

BS EN 450-1:2005+A1:2007 was recently published to supersede BS EN 450-1:2005 and BS 3892-1:1997, which are withdrawn. Fineness represents the main property change in BS EN 450-1:2005 and BS EN 450-1:2005+A1:2007 with up to 40% fly ash permitted as compared to the 12% value that UK (and elsewhere's) practice had been built on. In addition, an increase in LOI to up to 9% is introduced in the latest European standard presenting significant change. Designers are less restricted to specify PFA as a wide range of material quality is allowed in concrete. However, most previous research, practical experience and impressions are based on use of fly ashes created from earlier generation power stations; those ashes were both coarser and higher in carbon content than those available nowadays, and were therefore much less effective as pozzolans.

In addition to Europe, the USA, and Canada, the utilisation of PFA at different levels in concrete is permitted nowadays almost all over the world. For example, in Hong Kong, PFA content for 25-50% by mass of cementitious materials for un-reinforced concrete, and 15-25% for reinforced sections are prescribed for the new specification published by the National Transportation Department in 2000 (Qin 2004). Indian Standard IS: 456: 1987 permits 15-25% PFA as a replacement for cement in concrete (Rafat 2003a).

(b) PFA-cement reaction and strength development

The aim behind reviewing the PFA-cement reaction in this section is first to give the reader a concise background about how PFA reacts with cement (or more precisely the reactive silicon dioxide in PFA with calcium hydroxide liberated from cement's reaction with water) and to show how PFA contributes to concrete strength development.

Literature shows that for ordinary concrete (without PFA), when water is added to cement the reaction of water with cement takes place; compounds of calcium, silica and alumia known as calcium silicates (CS) in cement react with water to produce hydrated cement paste; this past contains nearly 70% calcium silicate hydrate (CSH), 20% $Ca(OH)_2$ (hydrated lime or simply lime), 7% sulphoaluminate, and 3% other secondary phases (Oner *et al.* 2005). In a simple reaction equation, (C_3S, C_2S) + H_2O = CSH + $Ca(OH)_2$. CSH is the gel (glue) that holds the aggregates together and gives dry concrete its strength and durability. However, $Ca(OH)_2$ is a very brittle crystal that makes a weakened matrix. When concrete is still fresh, $Ca(OH)_2$ is dissolved in water and because of bleeding and the wall effect where water collects around and under the particles. This makes for a weak zone of brittle crystals, known as the transition zone of about 30-50 μm thick (Illston & Domone 2001), formed between every aggregate surface and the surrounding matrix of the cement paste.

When a source of reactive silicon dioxide (SiO_2 or simply silica) such as fly ash is added (chemical requirements for fly ash as per BS EN 450-1:2005+A1:2007 dictate that the content of the reactive silicon dioxide shall not less than 25%), the concrete quality will most likely be enhanced due to two possible mechanisms: the first is physical; fly ash particles fill in the voids between cement and aggregate grains making it much harder for water to move, and the second is chemical; the reactive SiO_2 (silica should be amorphous, or glassy to be reactive) will react with $Ca(OH)_2$ in presence of moisture to produce more CSH, in simple form: $SiO_2 + Ca(OH)_2 = CSH$.

As a result of this pozzolanic reaction, larger pores are much reduced, the concrete matrix becomes denser and the concrete more strong and durable (Oner *et al.* 2005) this in turn will reduce its permeability and increase its resistance to the ingress of deleterious materials. In addition, the physical filling effect of the finer grains allows denser packing within the cement and reduces the wall effect in the transition zone between the coarse aggregates and the matrix (Isaia *et al.* 2003). The main results of pozzolanic reactions are: lower heat liberation and strength development, lower lime-consuming activity and smaller pore size distribution (Mazloom *et al.* 2004).

According to (Mehta & Aïtcin 1990) the small particles of pozzolans are less reactive than Portland cement, but when dispersed in the paste, they generate a large number of nucleation sites for the precipitation of the hydration products of the cementitious paste.

However, the pozzolanic chemical reaction is slow and that part of the strength gain coming from chemical reaction is developed over longer period compared with that of conventional concrete. But the physical filling effect, in contrast, is immediate. On the other hand, in many cases, concrete does not need full strength for several months and project specification can be adjusted to take this delay into account.

(c) PFA and the heat of hydration

It is well known that when the water is added to the concrete mixture, a chemical reaction between water and the silicates and aluminates takes place to form a product of hydration that results in solid concrete. This reaction will give off heat thus making it exothermic. The rate, at which this heat is liberated, is largely affected by the temperature at which the hydration process occurs. Basically, if the temperature around the concrete after placing is cooler than normal, then the rate of hydration will increase because it can dissipate the heat produced.

If the surrounding temperature is greater, the rate of hydration slows down because chemical reactions can not expel the heat created. The concrete's peak temperature will be significantly larger than the temperature of the cooled mass and therefore will create a larger range between the expansion and contraction of the concrete, therefore increasing the risk of cracking.

The replacement of cement with fly ash reduces the peak temperature the concrete will reach by not adding to the heat of hydration, therefore reducing the difference between maximum and minimum temperatures and thus lowering the risk of cracking (Neville 1995).

(d) The use of PFA in concrete

The use of PFA and other pozzolanic materials for NAC concrete in the past both in research and practical applications was very well established and widely recognized. Pozzolanic materials are defined as siliceous or siliceous and aluminous materials which in themselves possess little or no cementitious value but will, in fine particle form and in the presence of moisture, chemically react with calcium hydroxide at ordinary temperatures to form compounds possessing cementitious properties (ASTM C 618 1997).

(Sear 2005) reported that PFA has been extensively researched over a period of more than 50 years and found to be beneficial in resisting many deleterious reactions as well as improving the durability of concrete. In contrast, few research projects have been able to provide evidence regarding the effect of mineral admixtures on the properties of recycled concrete (Gonzalez & Martinez 2008); in most cases PFA was used to partially replace cement in mortar or concrete, however in a few research cases PFA was also used to substitute fine aggregates.

(i) PFA replacing cement

Cement is the most cost and energy-intensive component of concrete. The unit cost of concrete can be reduced by partial replacement of cement with fly ash (Rafat 2004). This section will briefly cover the use of PFA in ordinary and high strength concrete

(HSC). The aim is just to show how beneficial PFA is, and to learn how the expertise followed, particularly in creating HSC, could be utilised in achieving the target of this study; *i.e.* producing good quality RAC. A number of studies have been undertaken to produce different types of concrete containing PFA. However, the vast majority have used NA, and therefore most of the knowledge incorporating PFA was built based on the characteristics of NAC concrete mixes.

(Mehta 1985) had studied the use of different fly ashes in concrete (made with natural aggregate) from various sources in the US and found that the calcium content in fly ash and the particle size distribution were the most important parameters that control the rate of development of concrete strength. Because fly ash proved to be effective for preventing disruptive alkali silica reaction (ASR), it was recommended (BS 3692-1:1997) that a minimum of 25% fly ash is needed to prevent ASR. For coarser fly ashes, a minimum of 30% fly ash is required to ensure sufficient surface area to avoid ASR.

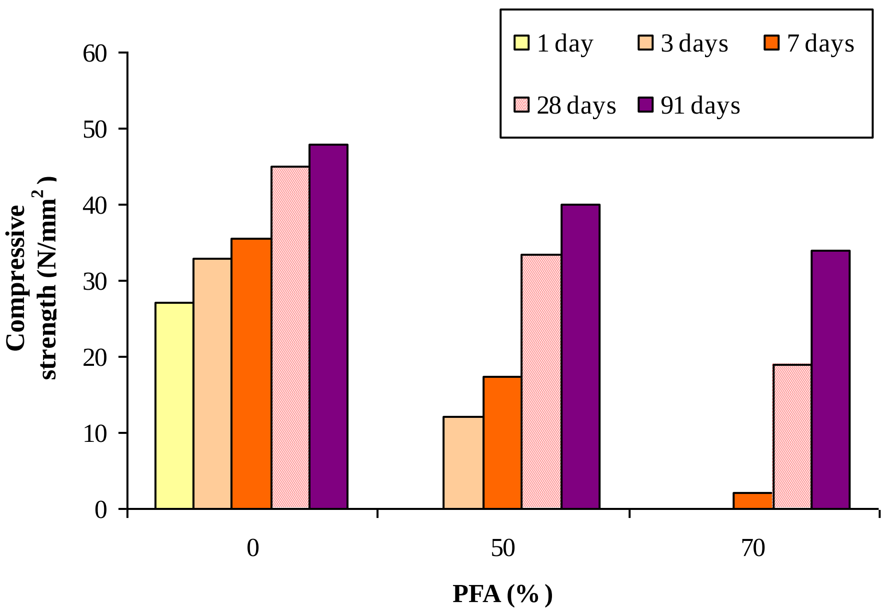


Fig. 3.2 Variation of compressive strength with the level of PFA and age (Naik *et al.* 1994)

(Naik *et al.* 1994) has concluded that concrete mixtures with up to 50% PFA replacing cement were able to attain sufficient strength. In his study it was reported that many other researchers had confirmed the use of high-volume PFA to produce concrete for structural applications. Typical variation of compressive strength of NAC with PFA at different amount and ages is shown in Figs 3.2 and 3.3.

Figure 3.2 shows that NAC concrete with 50% PFA binding material produces comparable strength at later ages; compressive strength is about 15% less than the control mix strength. With 70% PFA replacement, strengths are lower, particularly at early ages. More recently (Josette *et al.* 2006) are in agreement with results of the above mentioned investigation (Naik *et al.* 1994), Fig. 3.3 shows a similar chart to that shown in Fig. 3.2.

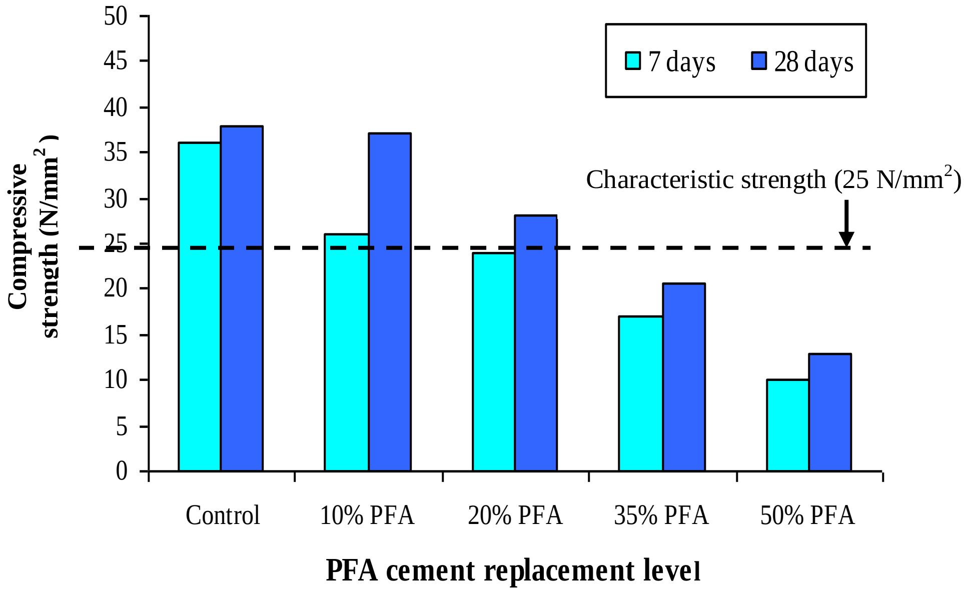


Fig. 3.3 Compressive strength of NAC concrete with various levels of PFA (Josette *et al.* 2006)

Concrete with 20% PFA addition was considered acceptable as it has achieved the characteristic strength after 28 days' curing, taking into account the tendency of PFA concrete to improve at later ages.

In another extensive laboratory study (Oner *et al.* 2005), a total of 28 different mix designs including PFA were tested, percentages of 15, 25, 33, 42, 50 and 58% of the cement content were added as a partial cement replacement. The results showed that strength increases with increasing amount of PFA up to an optimum value beyond which strength starts to decrease with further addition of PFA. The optimum PFA content was found to be 40% of cement. (Josette *et al.* 2006) reported that PFA reduces ASR when used with reactive aggregate.

It was reported (Malhotra 1990) that adequate concrete strength and excellent durability can be produced typically by maintaining the w/b ratio at about 0.32 with large dosages of superplasticizers. This study indicates that the main concern of using PFA in concrete is the rate of strength gain; the slow rate is the result of the relatively slow pozzolanic reaction of PFA. However, the rate of strength gain could be radically improved by the use of early strength cement or by blending OPC with chemical activators such as sodium sulphate anhydrite when higher early strength concrete is desired (Malhotra 1990).

A study (Tarun *et al.* 1996) reported that the results of concrete cast to contain 35% PFA cement replacement exhibited lower compressive strength, higher flexural strength and comparable modulus of elasticity of concrete at all ages as compared with the reference mixture containing no-fly ash. Blended class F and class C fly ashes showed better results than individual use (unblended).

PFA can also be used in the production of high strength concrete (HSC). HSC is simply ordinary concrete of excellent strength, durability and optimum lifecycle. Good quality natural or artificial aggregate were commonly used to produce HSC. The primary difference between normal strength and HSC relates to compressive strength. The meaning of the term high strength has changed significantly over the years: concrete having compressive strength more than 41 N/mm^2 was considered as high strength concrete (ACI 1989), later on, 50-80 N/mm^2 became viewed as high strength concrete, nowadays compressive strength in excess of 80 N/mm^2 is considered high strength concrete (Neville 1995); more details are given in Table 3.7.

PFA replacement can be as much as 50% of the cement content in ordinary concrete (Carette *et al.* 1993). For HSC, the ratio is limited to 15-25% (ACI 1987). HSC with PFA (ranging from 50-60% of the total binding materials) was produced by the Canadian Centre for Mineral and Energy Technology (CANMET) in 1985 (Poon & Wong 2000). However, others define HSC as that attaining compressive strength greater than that usually obtained in a region using locally available materials; as concrete currently produced varies considerably from one region to another (Beshr *et al.* 2003).
Pervious research findings demonstrated that the use of PFA and chemical admixtures were beneficial to concrete performance (Dhir *et al.* 1998). HSC is used in particular for the construction of high rise building and bridges. For the tallest ever building in the USA (The Sears Tower at 295 m) in Chicago, concrete with compressive strength up to 41 N/mm^2 was used (ECOBA 2006).

Relatively expensive mixes were presented in the literature; in most cases, mixes were observed to contain high doses of superplasticizer and other chemical additives, ultrafine powders, or a composite of such material. The aggregate and concrete were sometimes treated by chemicals and energy-consuming methods; as an example (Guangcheng *et al.* 2002) reported that very high performance concrete with ultrafine powders was produced by binary and/or ternary systems. Concrete matrices comprising PFA, silica fume and pulverized granulated blastfurnace slag were produced to attain compressive strengths up to 200 N/mm^2. Quartz sand of maximum diameter less than 0.63 mm, high performance steel fibres, SP, very low w/b ratios (0.154 to 0.18) and heat-treated samples were all utilized in the systems. In a similar manner, (Liu *et al.* 2000) stated that a composite of ultrafine fly ashes mixed with high dosage of SP and superior quality natural aggregate can be proportioned to achieve about 95 N/mm^2 when the PFA level of cement replacement was 35%. However, such special mixture concretes can be produced for particular purposes, but cannot be employed for RAC concrete as they can have limited practical application and are most unlikely to be cost-effective.

Studies in which RA was used to produce HSC are extremely few, probably because there are concerns about the strength of these aggregates. A study involving mixtures of PFA, OPC, GGBS and SP to investigate the properties of HSC using RA has been carried out (Tsung *et al.* 2006). Coarse and fine aggregates were produced from earthquake-demolished structures processed and sieved in a laboratory to meet the ASTM C33 standards. It was reported that recycled aggregates are not suitable for use in HSC due to their high absorption capacity, unstable properties, and relatively weak strength. The most important findings of this study revealed that many inadequacies of RA can be overcome through careful examination of the characteristics of RA and the adoption of proper mixture proportions.

Literature also shows that other cementing materials possessing pozzolanic properties such as natural pozzolans, ground granulated blastfurnace slag, silica fume, rice husk ash, metakaolin, and others were used either for research or practical purposes to substitute cement or as supplementary materials. However compared to PFA, the availability of these materials is rather limited, and they need higher doses of superplasticizer in concrete due to the very high fineness of particles such as is the case with silica fume, and those possessing higher porosity such as rice husk ash, palm oil ash, *etc.* (Vanchai *et al.* 2007). In general, it is noticeable that the mechanical properties are appreciably adversely influenced for high PFA replacements, although SP and other additives can have significant ameliorating impact.

(ii) PFA replacing fine aggregate

Although a number of significant results have been reported on the use of PFA as cement replacement in many practical and research works, there is not much literature available on the utilization of this material as partial substitution for fine aggregates. However to date, few studies had been carried out. That is almost certainly because fine aggregate is an abundant material and cheaper than PFA. However this is not always the case, different classes of fly ashes are also available in large amounts at very low cost or sometimes for free, and could be used as an alternative to sand or to enhance cementitious content; otherwise they are dumped in landfill sites.

In a study by (Rafat 2003a) in which NA was used, fine aggregate was replaced by class F fly ash at levels of replacement of 0, 10, 20, 30, 40 and 50% by weight. The corresponding w/b ratios were 0.47, 0.43, 0.38, 0.34, 0.32 and 0.29 respectively. The cement content was kept constant at 390 kg/m^3 for all mixes including the control (no fly ash) mix while water and SP were used at different dosages to maintain workability. It was concluded that the compressive strength, splitting tensile strength, flexural strength and modulus of elasticity of concrete specimens continued to increase with time, and were higher than the control mix at all ages. The corresponding 28 day strengths are 26.4, 28.2, 30.5, 34.9 and 40.0 N/mm^2; the elastic moduli ranged from 20-24.5 GPa. However the workability of concretes was lowered from 100 mm for the reference mix (without fly ash) to 20 mm slump for 50% replacement. It has been mentioned earlier in this study that fly ash is available free of cost in India and it may only involve transportation cost and does not incur any additional cost in making concrete as money will be saved on reduced use of fine aggregate.

Similar work (Rafat 2003b) was carried out to investigate the effect of fine aggregate replacement with class F fly ash on the abrasion resistance of concrete. In this study fine aggregate was replaced by class F fly ash at levels of replacement of 10, 20, 30 and 40% by weight keeping the cement content constant for all mixes at 370 kg/m^3. Results show that the compressive strength of concrete increased with the increase of replacement level, though decreased workability ensues. The abrasion resistance increased with compressive strength, age of concrete and with the increase in fly ash content. It was concluded that class F fly ash could be suitably used as partial replacement of fine aggregate in concrete. This study showed that 30% fly ash replacement produced the maximum compressive strength; Fig. 3.4 was reproduced from this paper.

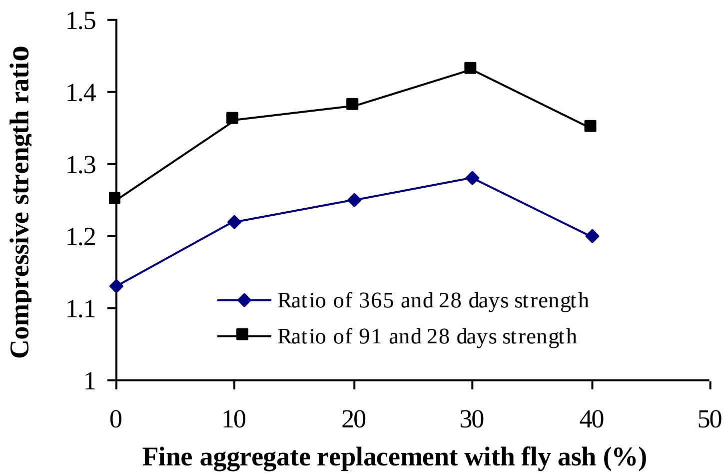


Fig. 3.4 Compressive strength ratio *versus* fine aggregate replacement with fly ash (after Rafat 2003a)

The wear and abrasion resistance of concrete is of importance when sewers and drains are considered. Campbell's research (Campbell 2008) highlights the importance of Horszczaruk's work attempting to model the abrasive wear of concrete (Horszczaruk 2008). It is believed that a lower w/c ratio increases wear resistance and vice versa. This was confirmed elsewhere (Ghafoori & Sukandar 1995; Sebök & Stráněl 2004), and in general, for w/c ratio (C) the volume wear rate in terms of mm^3/s, V_r is given by:

$$V_r = \text{C}[(t+1)]^{aC^b} \qquad \text{(Eqn 3.1)}$$

Constants *a, b* may be determined experimentally. The effects of RA and/or PFA have yet to be considered in this field.

In other research (Tahir & Khaled 1996), fine aggregate was replaced by crushed limestone dust at levels of 5, 10, 15, 20, 25 and 30%. This dust is collected from crushed stone sieved to pass the 75 µm BS sieve. The effects of various proportions of dust content on properties of fresh and hardened concrete were evaluated. The results show that as the replacement percentage increases, slump decreased, compressive and flexural strengths decreased beyond 10% replacement, resistance to impact was reduced beyond 5% replacement, absorption increased for concretes containing more than 15% replacement, and water permeability also decreased.

Other waste materials were used to replace fine aggregate in concrete mixtures, for instance, a study (Ibrahim *et al.* 1996) was conducted to evaluate the effect of using solid waste materials as substitutes for fine aggregates on the mechanical properties of cementitious concrete composites. It was concluded that waste materials such as plastics, glass, and fibreglass can be used in cementitious concrete composites without seriously hindering their mechanical properties up to the composition range used in this study (up to 20%). It was confirmed that the use of granulated plastics, glasses, and fibreglass waste materials is, indeed, a viable solution to recycling such materials in concrete composites. In another study (Bashar & Ghassan 2008), fine aggregate was replaced by recycled glass; it was reported that glass powder enhanced the plastic properties of concrete, but coarser glass reduces the consistency, causes severe bleeding and segregation.

However it is well known that higher content of fine materials (*i.e.* dust) in the aggregate increases the fineness (percentage of particles retained on BS sieve size 45 μm) and the total surface area of aggregate particles. Aggregates with higher specific surface area (the total surface area of all the particles to their volume) require more water in the mixture to wet the particles surfaces adequately and to maintain workability, but increasing water content in the mixture will adversely affect the strength and durability of concrete. The positive influence of crushed rock is physical; the dust behaves as filler and helps to reduce the total void content in concrete and thus makes denser and less permeable concrete.

When PFA is used as fine aggregate replacement at 30%, strengths comparable to control concrete were attained at 7 and 14 days, provided the cement proportioning is kept constant (Josette *et al.* 2006). Fly ash was used as partial replacement or as a fine aggregate in concrete because it is readily available locally in many places, proven to be efficient and compatible with PC and fine aggregate, yields improved mechanical properties and reduced water absorption of concrete (Jane *et al.* 2004).
The concentration of fly ash in PC composition produces an increase in compressive strength (Jane *et al.* 2004); previous SEM studies showed that the addition of fly ash to concrete mixes results in reduced voids due to the small and regular spherical shape of

fly ash grains; better packing of the aggregate-fly ash mixture resulting in increased strength.

An investigation (Maslehuddin *et al.* 1989) was undertaken to study the effect of fine aggregate replacement on the early age strength gain and long-term corrosion resisting characteristics of fly ash concrete; fly ash was used at replacement levels of 0, 20 and 30% and w/c ratio of 0.35, 0.4, 0.45 and 0.5, and with constant cement content at 350 kg/m^3. The investigation reveals that the compressive strength and corrosion resistance was higher for fine aggregate replaced with fly ash. Also, the corrosion rate of reinforcing bars was lowest at 30% replacement level.

(Dhir *et al* 2000) studied concretes with constant w/c ratio of 0.5 using three different natural coarse aggregates in which 5, 10, and 15% PFA replaced sand. Significant improvements of compressive strength at all ages were reported even though the workability of mixes was affected.

(Josette *et al.* 2006) showed that when fly ash was used as sand replacement, strengths comparable to control concrete were attained at 7 and 14 days, irrespective of the degree of replacement, as long as the cement proportioning remains constant. The best proportioning to obtain strengths comparable to the control mix at 28 days seemed to be 10% replacement of cement and 30% replacement of sand. This study concluded that the utilization of PFA for construction purposes is economical as it can replace both cement and fine aggregate producing good quality concrete.

A recent study (Rajamane *et al.* 2007) reported that when fly ash replaced sand at between 0 - 30% at w/c of 0.6 for ordinary concrete, the compressive strength was found to be enhanced, while a need arose to increase water content to compensate the adverse influence of the increased level of PFA on workability.

Recently, similar research (Aggarwal & Gupta 2007) was carried out to study the effect of bottom ash as replacement of fine aggregate in concrete; fine aggregate was replaced by 0, 20, 30, 40 and 50% bottom ash and SP at levels of 0.47, 0.63, 0.66, 0.70 and 0.75% of the total cementitious material, while the cement and water content were kept constant (cement content was 426.7 kg/m^3, w/c = 0.434). The study showed that as the level of bottom fly ash content decreased, the workability, compressive, splitting tensile and flexural strengths at 28 days decreased; however at later ages (90 days) with mixes that contain 30 and 40% ash, the strengths were 108, 105 and 118% of the 28 day compressive, splitting tensile and flexural strength respectively.

(iii) PFA replacement level

Literature has shown that many factors influence the behaviour of fly ash concrete mixes such as the type and composition of fly ash, the workability when PFA is added, the admixture, the target strength, and the earlier local experience of using a specific type of PFA *etc.* Depending on these factors, the ratio of PFA replacement to cement or fine aggregate can be different. For instance, early efforts for use of fly ash in concrete such as (Swamy *et al.* 1983) developed many mixes of normal and lightweight concrete containing natural aggregates and 30% fly ash by weight of cement which produced adequate strengths compared to the concrete without fly ash. Also, ACI Committee 226 (ACI 1987) reported that fly ash replacements of cement were mostly limited to up to 35% due to the requirement of additional water for wetting the finer fly ash which in turn increases w/c ratio and reduces the strength. Trial mixes are recommended by many workers for the optimum use of local materials.

A study (Ganesh & Rao 1993) demonstrated that most concretes with replacement up to 35 % showed strengths higher than the corresponding control, while the higher percentage replacements (ranging from 50 to 75%) showed a reduction depending on the percentage replaced and the w/b ratio. The aforementioned study also presented the results of twelve previous studies in which different types of natural aggregates and fly ash were used at different percentages of fly ash replacements; these studies showed that fly ash replacement ratios ranged from 15 to 75% of the cement content.

Research (Poon 2004) on high strength concrete prepared with large volumes of low calcium fly ash, reported that concrete with a 28 day compressive strength of 80 N/mm^2 was produced with 45% PFA replacement of cement and w/b of 0.24. The study also showed that compressive strength above 70 N/mm^2 can be produced with replacements up to 25% fly ash. The amount of fly ash in typical concrete applications ranges from 15-35% by weight, with amounts of up to 70% by weight for mass concrete in dams, walls, girders and for roller compacted concrete pavements and parking areas (Oscar 1999).

Concrete mixes with up to 30-70% cement replacement with PFA performed better than control concrete in sea water without serious deterioration (Memon *et al.* 2002). Concrete made with 40% PFA replacement of cement by weight was pumpable and gaining strength over time (McCarthy & Dhir 2005; Josette *et al.* 2006); it has been concluded that PFA levels of around 30% in cement have become generally accepted.

In BS EN 450-1:2005+A1:2007 or BS EN 450-1&2:2005, the level of PFA in concrete was not specified whereas older codes limited the level, for instance BS 6588 specified 35% maximum. However, the use of additions was specified in BS EN 206-1-2000 (Concrete - Part 1: Specification, performance, production and conformity) in which it was confirmed that k-value concept (to use w/b ratio where b = cement + k × addition) may be applied to fly ash conforming to BS EN 450. The maximum amount of fly ash to be taken into account for the k-value concept shall meet the requirement: fly ash/ cement ≤ 0.33 by mass.

(Josette *et al.* 2006) concluded that the best proportioning to obtain strengths comparable to control concrete at 28 days seemed to be 10 and 30% replacement for cement and fine aggregate. (Dhir *et al.*1998) have reported that almost in all respects, similar or enhanced performance to that of Portland cement concrete with levels of PFA at up to 45% is achievable. A replacement level of 25 to 35% forms the significant majority of PFA concrete produced; (Dhir 1998; Sear 2005) reported that experience with fly ash in structural concrete has generally seen the material used at levels of around 30% of the binder content. At this level, it was demonstrated that similar performance compared to that of Portland cement concrete of equivalent strength is possible. Beyond this level (*i.e.* higher levels) early age strength of concrete may be reduced which could have practical implications.

More recently, (Mouli & Khelafi 2008) reported that most pozzolanic materials tend to increase the mixing water requirements for concrete and lower the rate of strength development, and therefore, their proportion in blended Portland cements is generally limited to 30%.

A number of experts argued that 30% replacement is the most one should ever use, while others suggested 40 to 50% replacement level to get the greatest benefits of the high volume fly ash and in others 80-90% replacement is possible (Bruce 2006).

Several completed research and implemented projects have confirmed the possibility of producing high performance concrete with high volume PFA (50-70%), but the vast majority support its use for other concreting purposes; high volume PFA concrete is usually preferred for concrete designed to attain lower strength and whenever the environmental impacts are considered; as a larger amount of fly ash is used.

Close inspection of the published literature reviewed here and elsewhere, shows it is reasonable to conclude that the recommended replacement level ranges from 0-75%. However in most cases 20-50% PFA of cement or fines respectively seems to be more common. A 30% average value is most likely to bestow the advantages conferred by PFA and most unlikely to significantly affect the strength, particularly at early ages. This level of PFA is expected to contribute to increased strength at later ages as a result of pozzolanic activity.

From the discussion above, it is clear that PFA is added in various proportions depending upon the application. Based on this literature review, Table 3.6 can be established:

Table 3.6 PFA replacement level in concrete

Replacement Level	Application
Up to 5% as permitted by BS EN 197-1 (Very low level replacement)	Added to many types of cement as a minor additional constituent for industrial purposes.
5-20% (Low level replacement)	Commonly used in ordinary concrete and cement manufacturing.
25-35% (Common level of replacement)	Good quality concrete, this range forms the majority of PFA concretes produced worldwide.
35-50% (Moderate level of replacement)	Where low heat of hydration needed, and where there is a significant risk of ASR, chloride ingress or similar.
More than 50% (High volume PFA)	Concretes designed primarily to reduce the environmental impact of fly ash.

However, it should be noted that the particle density of fly ash is typically 2300 kg/m^3, significantly lower than for Portland cement at 3150 kg/m^3. Therefore, for a given mass of cement, a direct mass substitution of fly ash results in a greater volume of cementitious material in the mix. The volume is lower when fly ash is substituted for sand.

3.10 CLASSIFICATION OF CONCRETE

Literature showed that the compressive strength of concrete is the basic property used to classify concretes in most of the codes. Based on literature review, the following classification can be made.

Table 3.7 Classes of concrete based on compressive strength

Type of concrete	Range of compressive strength (N/mm^2)
Ordinary Concrete (OC)	Up to 50
Ordinary Lightweight Concrete (OLC)	Up to 50
High Performance Lightweight Concrete (HPLC)	> 50
High Strength Concrete (HSC)	50-80
Very High Strength Concrete (VHSC)	80-130
Fibre Concrete Composites (FCC)	150-800
Reactive Powder Concrete (RPC)	200-800

However, other properties may be used to classify concretes.

Published data showed that the UK construction industry typically uses concretes in the compressive strength range 30-50 N/mm^2. HSCs used in practice have generally had strengths in the range 50-80 N/mm^2, with over 100 N/mm^2 achieved in some cases (Neville 2003). Higher strength is not required in many concrete applications. The medium range performance characteristics therefore may be produced using traditional and recycled materials.

Several concrete mix design methods were developed and the concrete ingredients are dependent on many technical and economical factors. In this context, literature shows that RAC can be designed in much the same way as NAC concrete mixes; guidelines similar to those by RILEM committee (RILEM 1994) re-introduced here were recommended:

- When designing a concrete mix using recycled aggregate of variable quality, a higher standard deviation should be employed in order to determine a target mean strength based on a required characteristic strength.
- When coarse recycled aggregate is used with natural sand, it may be assumed at the design stage, that the free w/c ratio required for a certain compressive strength will be the same for RAC as for conventional concrete.
- For a recycled aggregate mix to achieve the same slump, the free water content will be approximately 5% more than for conventional concrete.
- The sand-to aggregate ratio for RAC is the same as when using NA.

- Trial mixes are mandatory and appropriate adjustments depending upon the source and properties of the RA should be made to obtain the required workability, suitable w/c ratio, and required strength, of RAC.

As a rule of thumb, the following general classification is widely accepted for preliminary designs which are often supplemented by trial mixes, adjustments, and mix optimization.

Table 3.8 Concrete mixes commonly used for various type of work

Type of work	Mix proportion Cement:sand:aggregate (by weight)	w/c ratio
High strength concrete	1:1:2 and 1:1.5:3	< 0.5
General reinforced concrete	1:1.5:2 and 1:1.5:4	0.4 to 0.65
Low strength concrete	1:3:5 and 1:4:8	0.70 to 1.1

3.11 SUMMARY

Literature showed that most of the earlier research on RAC was carried out using recycled aggregates produced predominantly from crushing of laboratory prepared concretes, mostly crushed 28 days after casting. The major concern is that cement hydration's contribution to strength will continue for a longer period, after the 28 days, and will be assumed to benefit the case for RA. However, this aggregate may not be representative of real life conditions in which crushed concrete often has much longer life, perhaps hosted in different environments before collection, and containing different impurities. Nowadays RA is produced predominantly from old concrete structures in a recycling facility and, available in commercial amounts, is therefore less variable and is more suitable to be used to assess the performance of RA than that produced in a laboratory for most of earlier studies. Therefore in this study, this type of RA will be used to cover such a gap. Similarly fly ash used in earlier research was predominantly produced in accordance with old standards imposing certain limitations on its physical and chemical properties; some of these have been changed and the effect needs to be appraised. The use of mineral admixtures in RAC concrete is limited, despite the well known advantages when used for NAC concrete mixes; representing another gap in research. Research into new uses and improvement of performance of products processed from waste materials is continuously advancing to match modern society's need for safe and economic disposal. From the literature reviewed in this chapter, the following remarks are drawn:

(a) General remarks

1. The use of recycled aggregates and industrial by-products in concrete saves non-renewable natural resources and landfill space. The use of RA is influenced by availability, demand, engineering performance, and financial issues

2. Recycled aggregates have not had a good reputation in the past and there are many factors which contribute to limit the use of recycled materials in construction. The variability of aggregate itself and its properties, the high absorption capacity, the overall performance, and limitations set by specifications and different standards are the major obstacles.

3. There is a common global recycling policy emergent in the last 10-15 years and still popular today; many treaties are in effect such as the Kyoto Protocol (as of February 2009, 183 states have signed and ratified the Protocol).

4. Various contaminants must be minimized in recycled aggregates; degradable materials are a source of weakness, chloride ions initiate steel corrosion, sulphate reactions cause binder disintegration, and ASR may cause expansion and cracking of concrete.

(b) Remarks on use of RA in concrete

5. The utilisation of RA and other by-products such as PFA in new concrete is more sustainable than conventional concrete. However to date, RA is used mainly for non-structural and low-utility applications.

6. Although still limited, recycling of C&DW is finding more acceptance in the construction industry due to advances in recycling and concreting technologies.

7. Research showed that in terms of stress-strain relationship, RAC exhibits similar behaviour to NAC, therefore concrete structures can be designed according to the prevailing theories used to design NAC members.

8. Quality of RA and the level of replacement have an influence on mechanical and deformation characteristics of RAC. As the replacement level is increased, the strength and stiffness are decreased.

9. As the strength of RAC is not as high as that of NAC, the cement content needs to be increased by 20-30% for RAC mixtures to achieve similar compressive strength.

10. To produce RAC concrete with similar workability to NAC concrete, water content needs to be increased by 5-8% depending on the aggregate's absorption capacity; cement content must be also increased to keep w/c constant. Alternatively RA is pre-soaked, however if RA is mixed in a saturated state *e.g.* not in surface dry condition the mechanical properties may be significantly influenced. SP and/or PFA can purposely be used instead.

11. More research work on RAC is needed for better understanding of its behaviour, and to improve its performance and acceptance in the construction industry.

(c) Remarks on use of PFA in concrete

BS EN 206 for concrete specification and production, and the UK complementary standard BS 8500 allowed PFA as an addition in concrete. BS EN 450- Fly ash for concrete permits a wide range of fly ash fineness to be used in concrete. Fly ash is the most commonly available such mineral additive therefore it is inexpensive. Moreover, up to a certain level, past experience with the addition of fly ash did not result in increased water demand or SP to adjust the workability of ordinary concrete. Unlike most other mineral admixtures, such as silica fume, previous work showed that PFA addition for higher strength concrete, did not generally require a high SP dosage, and exhibited a lower rate of workability loss. PFA is used in concrete to achieve several objectives; most are practical such as improving workability, enhancing strength and durability. The advantages are:

12. PFA at a particular level of cement or sand substitution or as a supplementary admixture can add to concrete strength; PFA generally slightly reduces the early age strength of concrete, but strengths continued to improve over time. While strength may be one of the most important concrete properties that control its performance, other fresh and hardened properties are equally important.

13. PFA minimise segregation and bleeding, improves workability, and decreases the possibility of thermal cracking. PFA can be used as a cheap mineral admixture in concrete; therefore reduces demand for natural resources.

14. PFA reacts with calcium hydroxide $Ca(OH)_2$; the brittle weak material produced from cement hydration and transforms it to calcium silicate hydrate CSH; resulting in increased strength, reduced permeability and porosity of concrete. Therefore PFA improves durability.

15. PFA could replace 20-50% of cement or sand for ordinary concrete without significantly affecting the concrete properties, particularly when combined with low w/c ratio and SP at 30% is an average value. The level is usually limited to 20% for HSC. The replacement is limited to these levels due the requirement of additional water for wetting the finer fly ash which in turn increases w/c ratio and reduces the strength.

16. The key factor in producing good quality fly ash concrete is the use of water-reducing SP to maintain a low w/b ratio. The end product will also depend on the pozzolanic performance of fly ash as well as physical grain effects.

17. The use of chemical and mineral admixtures in RAC concrete has received little attention, both on research and practical scales.

18. The utilisation of RA and PFA to create new RAC concrete is environmentally superior and is likely to be possible from a technical point of view (as compared to previously successful NAC concrete).

CHAPTER 4
HYPOTHESIS, METHODOLOGY AND MATERIAL CHARACTERISATION

4.1 BACKGROUND

In this chapter, the author's hypothesis will be introduced, and the methodology presented. The bulk of this chapter covers the characterisation of the materials used in this study. Testing of materials was carried out in compliance with current British and European Standards. Details of the material testing and results from which the parameters are obtained will be given. Some parameters were used for comparative purposes while others were used for the design of the author's various concrete mixes.

However, it should be noted that to date, recycled aggregates are tested in accordance with standards originally prepared for natural aggregates; recently, recycled aggregates are included in BS EN 12620: 2002+A1: 2008; only a few aspects are covered, but it was stated that new test methods for RA are at an advanced stage of preparation, and until that time current relevant standards are allowed.

In this chapter, the compatibility of SP with the binding material and the effect of SP and PFA on setting time and water demand of the binding paste and loss of slump of the concrete were studied. The factors that influence the selection of the appropriate type of SP were investigated, and the process of selecting the best dosage of SP and the corresponding water reduction that produces the most favourable conditions with respect to the designed workability are also clarified.

4.2 HYPOTHESIS

The hypothesis is that a comparable workability, strength, durability, and therefore overall performance of RA will be achievable when admixtures such SP, PFA, and red granite dust (RGD) powder are used in proper proportions. Two mechanisms of improvement are possible; chemical in which chemical reactions result in more calcium silicate hydrates (CSH) in the concrete paste and improved pore structure, and physical in which the filling or refining nature of fine grains could increase the packing density of paste and concrete. These mechanisms are believed to result in improved performance of RAC concrete. It follows that the author's null hypothesis would be that RA use causes no statistically significant improvement over NA.

The experimental testing of RAC and similar NAC specimens, produced with these materials, cast and tested under standard laboratory conditions in accordance with the current standards will be undertaken to test this hypothesis.

4.3 METHODOLOGY

It was introduced earlier that the major objective is to investigate the potential for producing good quality recycled concrete by utilising aggregates derived from old concrete, and study its fundamental properties. The method to be followed in this investigation, to achieve this aim, is simply the effective use of inexpensive mineral admixtures such as PFA or RGD; exploiting their wide range of advantages in increasing the strength and durability of RAC. New RAC concretes will be made with mineral admixtures; namely PFA and RGD and chemical additives or more precisely, water-reducing superplasticizers. Both conventional and self-compacting concrete mixes will be examined.

Several researchers (Dhir *et al.* 1999; Wong 2000) have reported that fly ash concrete made with natural aggregates achieves better strength performance when produced with lower w/b. This approach is an attractive option to be followed in making concrete with recycled aggregate in place of natural aggregate. In this investigation, polymer-based modern SP to maintain the workability at low w/b ratio will be used. In addition to the dispersing effect, use of SP implies a reduced water content, a better hydrated structure and, hopefully, better mechanical properties of the RAC concrete.

4.4 MATERIALS USED IN THE STUDY

Materials used in this study are:

(a) Natural aggregate

Crushed granite NA of 20 mm size which was proven to produce excellent NAC, and is widely used in Scotland will be applied in this investigation. Granite is an extremely durable aggregate with high strength and superior quality. NAC will be used as control mixes.

(b) Recycled aggregate

Recycled washed aggregate of 20 mm size that required no extra processing will be used as it was supplied from the recycling plant. This aggregate had been processed (crushed and screened) at Bowhill Recycling Plant, Scotland, UK and prepared for different uses. However, this RA was observed to contain 6.6% impurities (as detailed in Chapter 4 Section 4.5 (a) iii) such as timber, metal, glass, paper, rubber and mortar *etc*. Throughout this study, impurities were not removed from the RA so that their effects on the characteristics of the produced RAC are included; this is to simulate the case of real conditions when RA is used to make RAC in site.

(c) Fine aggregate

Natural concrete fine aggregate of medium grading was used throughout all the experimental work. This aggregate is broadly used for concrete in Scotland.

(d) Cement

To omit influences that might result when different types of cement are used, only ordinary Portland cement (OPC) was used. The Blue Circle Cement (Procem) produced in Scotland by Lafarge Cement UK, which complies with BN EN 197-1: CEM I 42.5N will be used throughout this investigation. Procem is the most common type of cement used for making concrete, and proved to be suitable for use in general concreting and cement-related works. Procem is fly ash-free cement.

(e) Pulverized fuel ash

PFA that complies with BS EN 450- Parts 1 and 2: 2005 will be used as a mineral admixture throughout this study.

(f) Superplasticizer

In normal cement paste *i.e.* without superplasticizer, cement particles combined to form non-uniform larger masses (agglomerates) of cement particles which were usually predominant in the cement-water suspension. The addition of a superplasticizer causes them to disperse and repel each other, resulting in better workability (Neville 2003). Thus the basic advantage of superplasticizers is the creation of higher workability concrete that results in easy placement, without reduction in cement content and strength. Superplasticizers are broadly subdivided in four groups:

- Sulphonated melamine-formaldehyde condensates (SMF)
- Sulphonated naphthalene-formaldehyde condensates (SNF)
- Modified lignosulphonates (MLS)

- Others including sulphonic acid esters, carbohydrate esters, *etc.* Variations exist in each of these classes and some formulations may contain a second ingredient.

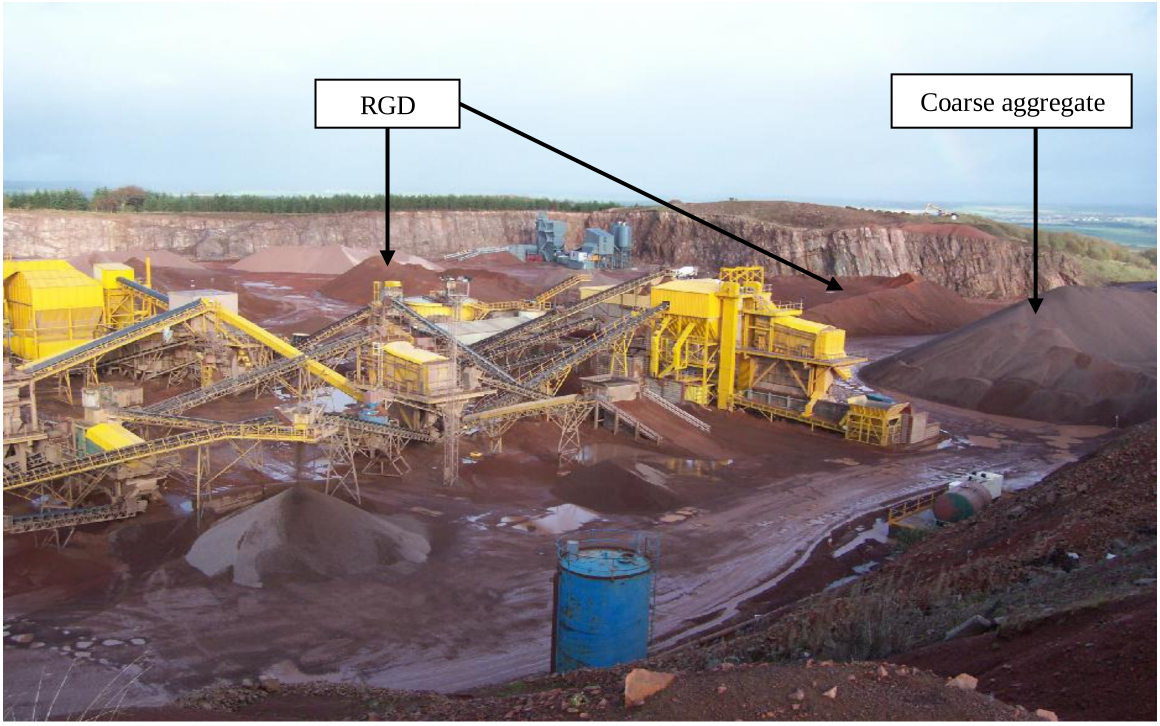
The dosage of superplasticizer should be adjusted to work with the concrete components effectively to avoid segregation, bleeding and slump loss. The incorporation of a greater dosage could be more advantageous, but this does not mean that excessive amounts can be always tolerated. Beyond a particular amount the excessive use of superplasticizer may lead to other adverse and undesirable properties with respect to concrete workability, segregation or bleeding, in addition to increased cost.

In this study, a suitable SP and dosages will be used. The type of SP will be selected from a list of three commercially available concrete superplasticizers. The appropriate type and dose will be selected on the basis of their compatibility with the binder (cement and PFA), workability of mixtures and mechanical performance

(g) Red granite dust

It was noted (Chapter 2 Section 2.2 (e)) that considerable amounts of this waste material go to landfill sites. One of these quarries, from which an RGD sample was supplied, is Cloburn quarry which is located near Lanark. The potential of recycling RGD in concrete will be investigated in Chapter 7. Fig. 4.1 shows a view of the quarry and an RGD sample.

(a) RGD sample



(b) Quarry view

Fig. 4.1 Cloburn quarry near Lanark (OS Grid ref. NS 935408)

(h) Water

It is well known that water has a great influence on the strength of concrete. The more water added, the weaker the concrete. A general rule for the quality of water in concrete is that if it is fit for human consumption it can be used for concrete.

When mixing water contains sufficient amounts of dissolved or solid impurities it must not be used for concrete as it will have various effects on both fresh and hardened properties of concrete. Effects on the setting, cement hydration, strength, promotion of different chemical reactions and corrosion to the reinforcement are examples. For the above mentioned reasons, excellent quality potable tap water will be used for concrete mixing all through this investigation.

4.5 MATERIAL TESTING

(a) Testing of aggregates

The testing of the different materials, which will be used to produce the concretes in this research, was undertaken to ensure they satisfy the requirements of current standards to investigate their various properties. Focus will be made on testing of the properties of aggregates, particularly the recycled aggregate. The aim is to assess the materials' suitability for purpose; the production of RAC, and to obtain the required mix design parameters. Results will be described in this chapter.

Physical and mechanical properties of the aggregates need to be investigated as these in turn will affect the properties of fresh and hardened concrete. The main aggregate properties to be considered when recycled aggregates are used in new concrete are: specific gravity, absorption, angularity, gradation, compressive, flexural, and tensile strength, susceptibility to alkali-aggregate reactivity and freeze-thaw distress (Gilpin *et al.* 2004).

To obtain representative samples, aggregates were riffled in compliance with BS 812: Part 102: 1989 "Method of Sampling", the sample is split into two equal portions to decrease the size to a practical amount while ensuring the sample is representative. The selected aggregate samples were then tested for grading, impact, relative density, water absorption, and porosity. The estimation of impurities in the recycled aggregate will also be given. The description of each test, the apparatus used, and the procedure are outwith the scope of this report and can be found elsewhere; for instance BS 882 for particle size distribution sieve analyses, BS 812 Part 2 for particle density and water absorption, BS 812 Part 112 for aggregate impact value, *etc*.

(i) Grading of coarse aggregates

Sieve analysis was carried out on all coarse and fine concrete aggregates before their use in the experimental work. The sieve analysis is used to find the amount of different size aggregate in a particular sample; it is carried out by putting the sample through a series of sieves that get progressively smaller. For control purposes all samples of aggregates were air dried for equal periods of time before testing. Suitable stacks of sieves were used for each analysis in accordance with BS 812: Part 103: 1989 and BS 410: 1986.

Table 4.1 displays the results of the sieve analysis for a sample of coarse aggregates used in the study, the results of the sieve analysis for NA and RA coarse aggregate were then compared with values listed in Table 4.2 to determine if the aggregates complied with the grading limits for 20 mm aggregates, extracted from BS 882-1983.

Table 4.1 Typical sieve analysis result of NA and RA coarse aggregates

Sieve size (mm)	Percentage by mass passing BS sieves for nominal sizes		Limits for single-sized aggregate (from BS 882 [111])
	NA	RA	
37.5	100	100	100
20	92.5	89	85-100
14	33	41.5	-
10	6.5	4.5	0-25
5	0	0	0-5
2.36	0	0	-

Table 4.2 Grading limits for coarse aggregate (from BS 882-1983)

Sieve size (mm)	Percentage by mass passing BS sieves for nominal sizes						
	Graded aggregate (mm)			Single-sized aggregate (mm)			
	40-5	20-5	14-5	40	20	14	10
50	100	-	-	100	-	-	-
37.5	90-100	100	-	85-100	100	-	-
20	35-70	90-100	100	0-25	85-100	100	-
14	-	-	90-100	-	-	85-100	100
10	10-40	30-60	50-85	0-5	0-25	0-50	85-100
5	0-5	0-10	0-10	-	0-5	0-10	0-25
2.36	-	-	-	-	-	0	0-5

The data presented in Tables 4.1 and 4.2 indicate that the tested NA and RA had sieve analysis gradings which placed them within the limits for 20 mm aggregates. Throughout this investigation, the same aggregate with almost the same grading will be used; it is well known that the use of aggregates with different grades could have significant influence on workability and strength of concrete, even when the same type (geologically speaking) of aggregate is used.

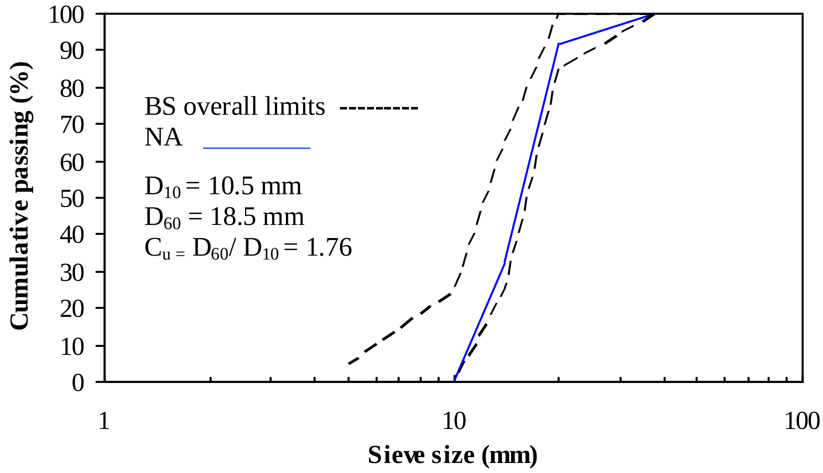


Fig. 4.2 Grading of natural coarse granite aggregate

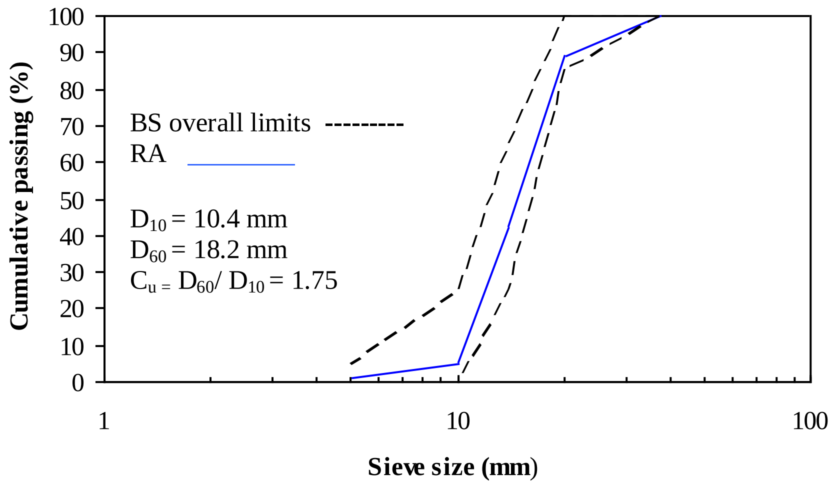


Fig. 4.3 Grading of recycled coarse aggregate

The uniformity coefficient (C_u) is the ratio of the grain size at which 60% is finer by mass to the grain size at which 10% is finer by mass from the grain size distribution curve. It is a measure of the uniformity of an aggregate; it shows how well or poorly sorted the aggregate is. A uniformity coefficient of unity denotes a material having all particle grains the same size; numbers increasingly greater than one denote increasingly less uniformity. The higher the number the more blended the coarse and fine elements of the aggregate. Result show that NA and RA have similar C_u.

(ii) Grading of fine aggregates

Natural fine concrete aggregate was used throughout the investigation. Table 4.3 gives sieve analysis test data of the fine aggregate sample. The grading limits according to BS 882-1983 for fine aggregates are shown in Table 4.4.

Table 4.3 Typical sieve analysis result of fine aggregate

Sieve size	Mass retained (g)	Mass passing (g)	Retained (%)	Cumulative passing (%)	% Passing (Overall limits from BS 882)
10 mm	0	551.24	0	100	100
5 mm	13.54	537.70	2	98	89-100
2.36 mm	92.64	455.06	17	81	60-100
1.18 mm	85.44	359.62	15	66	30-100
600 μm	67.74	292.08	12	54	15-100
300 μm	97.74	194.34	18	36	5-70
150 μm	87.24	107.10	16	20	0-15
Pan	106.34	0.76	20	0	
Total	550.48				
Loss (g)	0.76				
Loss (%)	0.14				

Table 4.4 Grading limits for fine aggregate (from BS 882)

Sieve size	Percentage by mass passing BS sieve			
	Overall limits	Additional limits for grading		
		Coarsest	Medium	Finest
10 mm	100	100	100	100
5 mm	89-100	89-100	89-100	89-100
2.36 mm	60-100	60-100	65-100	80-100
1.18 mm	30-100	30-90	45-100	70-100
600 μm	15-100	15-54	25-80	55-100
300 μm	5-70	5-40	5-48	5-70
150 μm	0-15	0-15	0-15	0-15

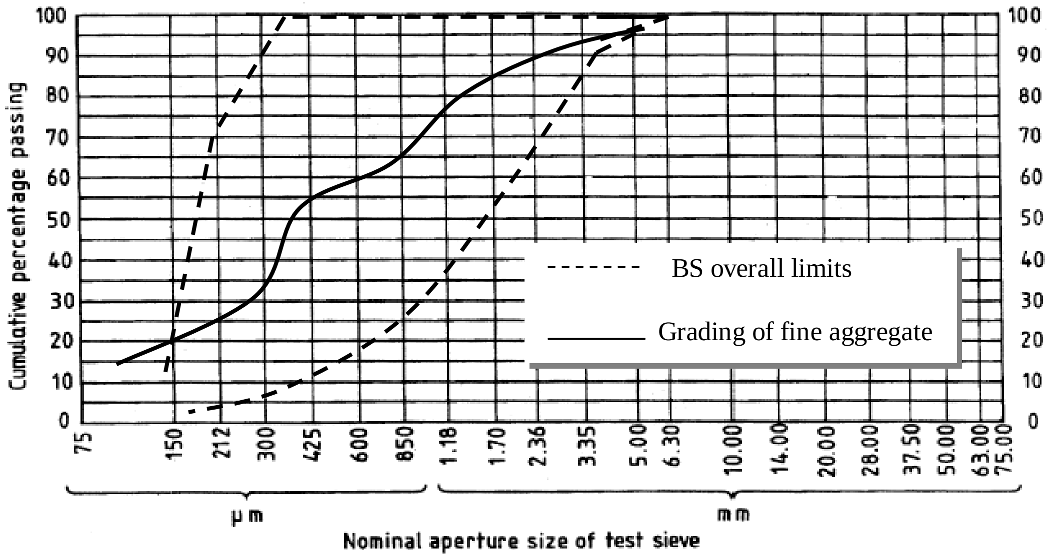


Fig. 4.4 Grading of fine concrete aggregate

The results are plotted in Fig. 4.4. The graph shows that this material is suitable for concrete as it fits the limits set out in BS 882-1983. However, for this sample 20% was outwith limits while the average of the tested samples was about 15%, such deviation would not have considerable influence on concrete properties. Several samples were tested and the average value for the percentage of fine aggregate passing the 600 μm sieve was 54%; this value will be used in future concrete mix designs.

(iii) Impurities in RA

Impurities are considered as sources of weakness of RAC concrete. For the RA used in this study, impurities were estimated roughly by visual inspection of their content. Four samples of the RA were tested and the individual content by mass was obtained and average amount of impurities content are obtained, results are displayed in Table 4.5. Fig. 4.5 shows a sample of RA used in this study; impurities are clearly seen.

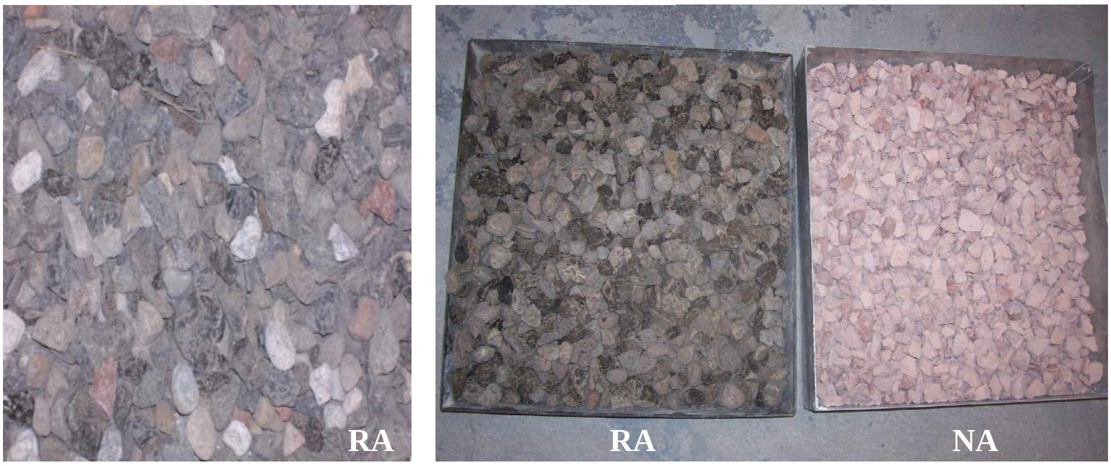


Fig. 4.5 Impurities in the RA

Table 4.5 Average percentages of impurities present in RA

Impurity	Average percentage by mass (%)
Asphalt	0.8
Glass	1.1
Ceramic	1.5
Metals	1.2
Timber	0.5
Others	1.5
Total	6.6

Values in the table are average readings from four samples.

The bond between the aggregate and cement paste is an important factor affecting the strength of concrete (Khalaf & DeVenny 2004). A quantity of the RA supplied was observed to be partially coated by bitumen. The presence of such aggregates in the mixture was expected to have an effect on the bond between the aggregate and concrete paste thus weakening it. Bitumen-coated particles were removed from four samples of the RA and weighed to determine their fraction in the aggregate.

Table 4.6 Typical result for the estimation of percentage of bitumen-coated RA

Sample no.	Sample weight (OVD) (g)	Bitumen coated aggregate (g)	Percentage bitumen coated (%)
1	1000	151	15.1
2	646	97	15.0
3	792	134	16.9
4	869	156	17.9
5	1041	174	16.7
6	935	168	17.9
7	970	138	14.2
8	814	142	17.4
9	974	149	15.3
10	725	119	16.4
Average			16.3

The results showed that average content of the bitumen-coated RA was about 16%, therefore, this could bring the total asphalt content up to 3-4% which is somewhat less than the 5% specified as the maximum limit in the current BS EN 8500-2. However the recently published European Standard; BS 12620:2002+A1:2008 limits asphalt content (Ra) to a maximum of 10% by mass.

The tested samples also showed the presence of about 14% of natural gravel aggregates, with this level, it is not expected that gravel aggregate will behave very differently to other aggregate derived from crushed natural rocks. RA also contains 3.4% brick aggregates. However, in BS EN 8500-2:2002 recycled aggregates containing more than 5% crushed brick are classified as mixed recycled aggregates.

The average total mass of impurities in RA tested samples accounted for about 7% of the total mass of aggregates. There was also a proportion of mortar present in this aggregate but it was difficult to estimate the actual percentage because the mortar mainly adhered to aggregate particles. Thus when the mortar is added, the total percentage of impurities in Table 4.5 would probably rise to 10-12%. This figures show that despite advances in RA processing technologies there are still impurities. Glass is not present in significant amounts; this is perhaps because it is common practice to remove glass from buildings to go into another recycling outlet before destruction. However the level of impurities in the tested RA samples, although acceptable, could have adverse impacts on the overall performance of RAC concrete.

In general, the higher level of impurities in RA is an indication of poor sorting before demolition of structures on site and before crushing of C&DW in the recycling facility; professional deconstruction and recycling results in lower impurity content.

(iv) Aggregate impact value

The aggregate impact value (AIV) is used to establish the material's ability to resist impact and assess the extent of particle crushing thereafter. The impact value is calculated by recording the fractions passing and retained in a 2.36 mm sieve after the material has received 15 blows from a standard weight. This is expressed as a percentage of the total weight. This test is carried out to measure the resistance of a particular aggregate to sudden shock or impact. AIV in accordance with BS 812-112:1990 were determined in a dry condition for all aggregates. A lower percentage indicates tougher and stronger aggregates.

The maximum allowable impact values for concrete aggregates given in BS 812: Part 112: 1990 are as follows:

- 25% when the aggregate is to be used for heavy duty concrete flooring
- 30% when the aggregate is to be used for pavement wearing surfaces
- 45% when the aggregate is to be used for other concretes

A sample of the aggregate impact test results for NA and RA are given in Table 4.7

Table 4.7 Typical impact value test for NA and RA

Type of aggregate	NA	RA
Mass of steel cup	2557 g	2652.6 g
Mass of cup and aggregate	3181.62 g	3238.8 g
Mass of aggregate (Mass A)	624.62 g	586.1 g
Mass passing 2.36 mm (Mass B)	26.22 g	66.2 g
Mass retained	598.01 g	519.16 g
AIV= (B/A) × 100	4.2%	11.3%

The AIV is determined as the percentage of the fraction of fines due to impact to the total aggregate; that is (B/A) × 100.

The average impact value of four specimens showed that the AIVs were 4.5 and 12% for NA and RA respectively. Based on the categories given in BS 812: Part 112: 1990, the aggregates fall within the suitability limits for concrete which can be used for heavy duty flooring and pavement wearing surfaces. However, good AIV is not the only parameter that guarantees good quality concrete. Table 4.7 also shows that the RA is weaker than NA but this was expected, mainly due to the presence of mortar that adhered to RA particles which resulted in an increased amount of fines obtained under impact.

(v) Unit weight of aggregates

Aggregates were tested under saturated surface dry conditions (SSD) in accordance with BS 812: Part 2: 1995 to measure the unit weight. Results showed that the average unit weights (or densities) were 25.5 kN/m^3 (2600 kg/m^3) and 24.5 kN/m^3 (2500 kg/m^3) for NA and RA respectively. As these densities are > 2000 kg/m^3, NA and RA were classed as normal weight aggregates. The relative density of a material is the ratio of its unit weight to that of water and it has a major influence on the density of the finished concrete. Most rock types have relative densities within a limited range of approximately 2.55 - 2.75, and therefore all produce concretes with similar densities, normally in the range of 2250 - 2450 kg/m^3, depending on the mix proportions (Illston & Domone 2001).

Because of the large content of old mortar in the crushed material, the densities of recycled concrete aggregates in most of the cases varies from 5 to 10% below than the density of the corresponding original aggregates (Hansen & Narud 1992).

To determine the specific gravity or relative density, the weight in air (A) and the weight in water (B) of the aggregate is obtained, then specific gravity is calculated as the ratio A/(A – B). The majority of natural aggregates have an average specific gravity between 2.6 and 2.7; lower aggregate density is usually a result of lower specific gravity (Neville 1995).

Table 4.8 Relative density of coarse NA

Property	Relative density (RD)		
	Sample 1	Sample 2	Sample 3
Weight in air (g)	2024.2	2114.6	2064.5
Weight in water(g)	1275.5	1320.7	1292.2
Volume (cm^3)	748.7	793.9	772.3
Average RD	2.704	2.664	2.673

Density of water 1g/cm^3

Average RD of NA = 2.68.

Table 4.9 Relative density of coarse RA

Property	Relative density (RD)		
	Sample 1	Sample 2	Sample 3
Weight in air (g)	2268.7	2225.5	2292.4
Weight in water(g)	1360.4	1348.2	1389.2
Volume (cm^3)	908.3	877.3	903.2
Average RD	2.498	2.536	2.538

Average RD of RA = 2.524.

Relative density is used to determine the equivalent weight of a material to a certain volume; for instance if a coarse aggregate of relative density 2.7 represents 60% by volume of concrete, the equivalent weight should be 2.7 × 60 × 9.81 = 1589 kg/m^3 ($W = mg = \rho vg$), where $g = 9.81$ m/s^2 is the acceleration due to gravity.

The literature (Turcotte 1993) explores the fractal relationship, or its possibility, between particle size and specific gravity; although an interesting diversion for the curious reader, this experimental work treats specific gravity as applying equally to all size fractions from the same crushed rock. The rounded average specific gravities of the tested aggregates were 2.7 for NA and 2.5 for RA; these values will be used for the author's mix designs herein.

Results showed that RA density is slightly lower than that of NA, probably because of the presence of impurities and old cement paste. However, when a material is partially substituted for another in a mix, the replacement by weight could result in a greater volume of the material in the mix if the difference between their specific gravities is large; therefore if the density of RA is much lower than NA, mixes should be proportioned to take this difference into account. In this study no partial substitution will be examined and 100% RA will be used. Density measuring apparatus is shown in Fig. 4.6.

Fig. 4.6 Density testing apparatus (weight-in-air, weight-in-water method)

The density testing apparatus consists of an electronic balance with a hanging basket attached beneath. Below this basket is a tank filled with potable water which can be raised and lowered. The aggregate sample is placed into the basket and weighed in air. The tank of water is then raised until the sample is completely submerged and the sample is re-weighed. The volume of the sample is derived from the difference between the weighing in air and water. Therefore, from these two figures the density for the virgin aggregate and the recycled aggregate are simply calculated. The specific gravity of the aggregate is calculated or obtained straight away from the scale knowing the density of water. Similar procedures were followed in determining the density of concrete cubes.

(vi) Porosity of aggregate

Porosity is a very important property of the aggregate as it affects the concrete in both the fresh and hardened states. The measure of how much open space exists in a solid particle is called its porosity. The porosity is expressed as a percentage of the total volume from 0-100%. If an aggregate has a high porosity, it can lead to weaker and less durable concrete compared to a concrete containing aggregate with lower porosity, especially if it is subjected to cyclic freezing and thawing.

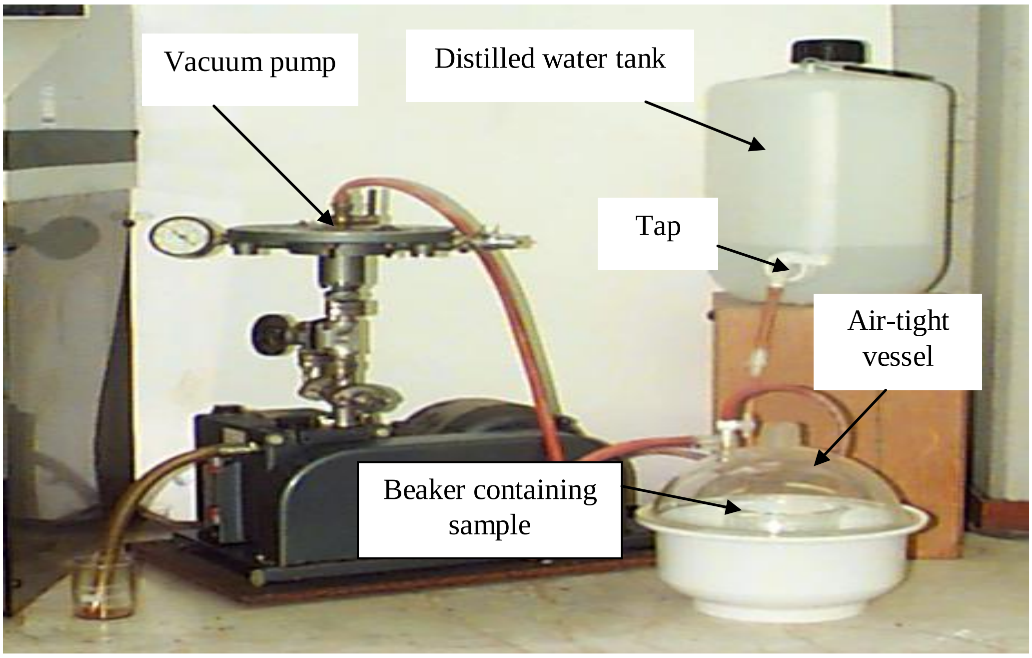


Fig. 4.7 Porosity testing apparatus

The test procedure (see Fig. 4.7) is detailed in BS 3921; reference can also be made to (DeVenny 1999; Khalaf & DeVenny 2002) for a modified test procedure. To determine the porosity of an aggregate its water absorption must be measured. This is the mass of water a dry aggregate absorbs until it reaches a saturated surface dry state, expressed as a percentage of the mass of dry aggregate. Recycled aggregates are inherently more porous, have high absorption capacity, and low specific gravity. These characteristics in turn could have a bearing on the behaviour of RAC in both fresh and hardened states (Bairagi & Kishore 1993).

Water absorption values were 6-7% by mass when RA was used at 20% by mass of NA, and 9% when used at 100% of NA (Levy 2004). Water absorptions of 5-10% can generally be found for recycled aggregates, however, relatively low absorption values were found for coarse recycled aggregates (Hansen & Narud 1992). Water absorption in RA ranges from 3-12% for coarse and fine fractions; this value is much higher than that of the natural aggregate for which the absorption is about 0.5-1.0% (Akash *et al.* 2007).

In these experiments, the average porosities of the aggregates were 15% and 18%; water absorptions were 2.85% and 8.50% for NA and RA respectively. The measured porosities were not much higher. When the porosity and the absorption capacity of an aggregate are high, it will have higher water demand as it would absorb a large proportion of the mixing water when used in concrete. In such cases, the aggregates can be pre-soaked and then used in a surface dry condition. Alternatively, the absorption capacity is determined and extra water added to compensate for the absorbed water so that the desired consistency of concrete can be produced. Both methods however need good quality control and entail impractical measurements. The use of SPs could do the job, abandoning the need to add extra water.

(vii) Drying of aggregates and absorption capacity of aggregates

To monitor the drying of aggregates and determine their absorption capacity, the initial water contents of six aggregate specimens were measured; three for NA and three for RA. The sizes of specimens were large enough so that several smaller samples can be taken from every specimen to measure the water content at different times. Once the initial water contents were obtained, specimens were laid to dry in air, under room temperature, in the same place where the aggregates used in this study will be dried *i.e.* inside the concrete laboratory. Samples from each large specimen were taken every hour to measure the water content. The process continued until constant, equilibrium water content was reached. An average value of three samples for each case was obtained. The absorption capacity after drying will be the difference between the initial and the final water content. Table 4.10 shows the results of tests carried out to obtain the initial water contents of aggregate samples.

Table 4.10 Initial water content of aggregate samples

No	Type of aggregate	SSD mass (g)	Oven dry mass (g)	Mass of water (g)	Water content (%)
1	NA	301.5	289.3	12.2	4.22
2	NA	302.5	293.0	9.5	3.24
3	NA	299.5	289.4	10.1	3.49
Average					3.70
1	RA	291.2	275.6	15.6	5.66
2	RA	315.8	298.1	17.7	5.94
3	RA	214.9	203.0	11.9	5.86
Average					5.82

Results of water content values at different times are presented in graphical form in Figs 4.8 and 4.9.

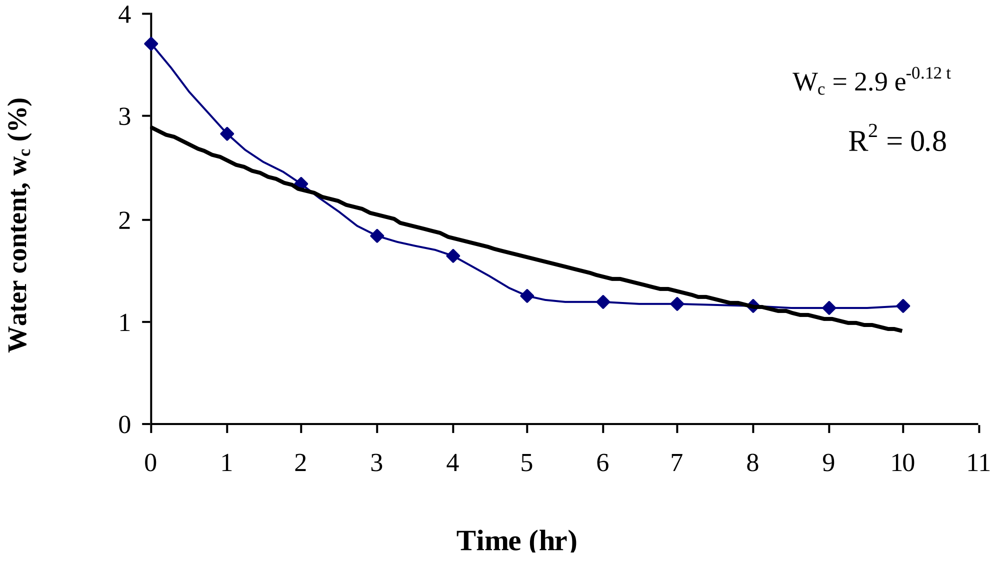


Fig. 4.8 Drying of NA over time

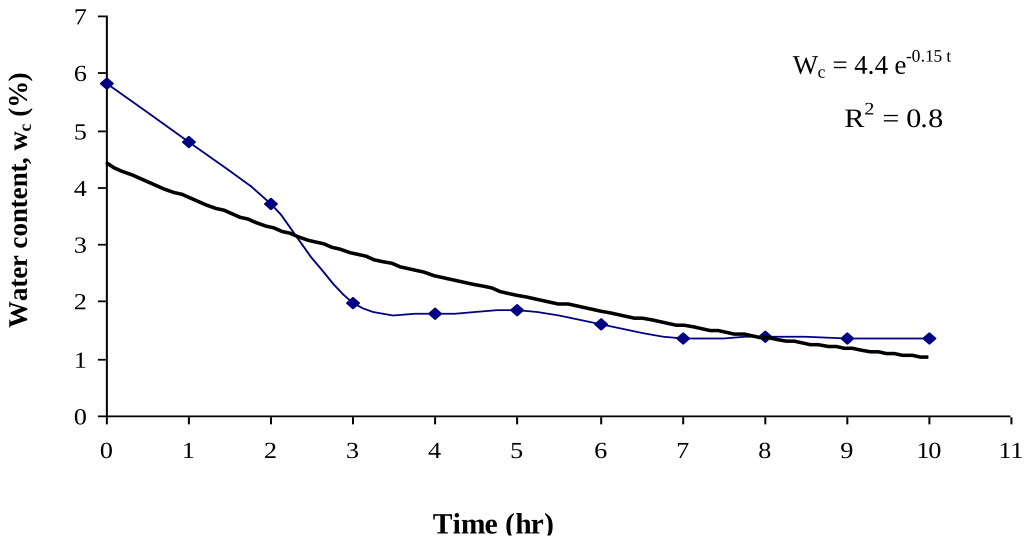


Fig. 4.9 Drying of RA over time

The tests show that water content reached its equilibrium value after 5 ¼ and 7 hours for NA and RA respectively, while the corresponding water capacities therefore are 2.6% (3.7 - 1.1 = 2.6 %) and 4.6% (5.8 - 1.2 = 4.6%). It was observed that both types of aggregate reached almost equal water content upon drying for significantly different periods of time at the same ambient temperature. The differing absorption capacity accounts for this. Therefore, to avoid pre-soaking or adding extra water to compensate for the absorbed water, as both methods are difficult to control, an SP can be used to cover the slight loss of free water in the mix due to the expected absorption. One can also see the initial plateau in drying curve of the RA; that is possibly due to variability of RA and the presence of contaminants which can hold moisture for some time.

(b) Testing cement, PFA, and RGD

As cement, PFA, and RGD powder are very fine materials having low solubility in water at 20°C, wet sieve analysis is usually carried out for their gradation. Representative samples dried in an oven at 115 ± 5°C were used to undertake wet sieve analysis to BS 1377:1990. Results were as follows:

(i) Wet sieve analysis of cement

Mass of cement sample sieved 101.97 g.

Table 4.11 Wet sieve analysis of cement

Sieve size (μm)	Mass retained (g)	Mass passing (g)	Percentage retained	Percentage passing
150	9.4	92.57	9.22	90.78
75	54.5	38.07	53.4	37.38
63	1.9	36.17	1.86	35.52
45	12.33	23.84	12.1	23.42
38	15.43	8.41	15.1	8.32
Total	93.56		91.68	8.32

Data in Table 4.11 show that 92% is coarser than 38 μm (8% is finer) and 12% was retained on the 45 μm sieve.

(ii) Wet sieve analysis of PFA

Mass of PFA sample sieved 102.46 g.

Table 4.12 Wet sieve analysis of PFA

Sieve size (μm)	Mass retained (g)	Mass passing (g)	Percentage retained	Percentage passing
150	3.65	98.81	0.36	99.64
75	57.7	39.11	57.43	42.21
63	0.5	38.61	0.5	41.71
45	10.55	28.06	10.5	31.21
38	13	15.06	12.94	18.27
Total	85.4		85	15

Data in Table 4.12 show that 85% is coarser than 38 μm (15% finer) and 10.5% was retained on the 45 μm sieve.

(iii) Wet sieve analysis of RGD

Mass of RGD sample sieved 126.03 g.

Table 4.13 Wet sieve analysis of RGD

Sieve size (μm)	Mass retained (g)	Mass passing (g)	Percentage retained	Percentage passing
150	0.8	125.23	0.63	99.37
75	22.5	102.73	17.85	81.52
63	5.3	97.43	4.21	77.31
45	23.6	73.83	18.73	58.58
38	8.1	65.73	6.43	52.15
Total	60.3		47.85	52.15

Data in Table 4.13 show that that 48% is coarser than 38 μm (52% finer) also 19% was retained on the 45 μm sieve.

Comparing the three materials on the basis of the percentage retained on the 45 μm sieve, the results showed that 10.5, 12 and 19% were retained thereon for PFA, cement, and RGD respectively. This however indicates that PFA was slightly finer than cement, and RGD is comparatively the coarsest, but with more uniform particle sizes as shown by its particle size distribution and uniformity coefficient when compared with cement and PFA.

(iv) Chemical analysis of cement, PFA and RGD

Chemical compositions of cement, PFA and RGD were obtained from the producers as shown in Table 4.14. Chemical analysis of an RGD sample was carried out by CERAM Building Technology's testing centre, Scotland, at the producer's request. Other properties of cement and PFA are shown in Tables 4.15 and 4.16.

Table 4.14 Chemical composition of cement, PFA and RGD*

Properties	Typical value (%)		
	Cement	PFA	RGD
Silicon oxide SiO_2	21.8	45.0	61.4
Aluminium oxide Al_2O_3	4.20	30.0	16.3
Iron oxide Fe_2O_3	2.50	11.0	3.66
Calcium oxide CaO	65.1	5.90	3.69
Magnesium MgO	-	2.25	1.70
Potassium K_2O	0.72	3.40	3.75
Sodium Na_2O	0.13	1.30	3.62
Sulphate SO_3	-	1.50	0.05
Na_2O_{eq} (Na_2O + 0.658K_2O)	0.60	3.51	6.09
Other oxides	3.15	-	1.10
Loss on ignition LOI	-	11.0	5.01

*values according to producer

Table 4.15 Other properties of cement*

Typical properties	Range
Surface area (m^2/kg)	300-400
Specific gravity	3.15
Initial setting time minutes	80-200
EN 196-1: Mortar-Compressive strength (N/mm^2)	
2 days	21-31
7 days	40-50
28 day	52-62
Bulk density (kg/m^3)	1300-1450
Sulfate SO_3 (%)	2.5-3.5
Chloride Cl (%)	Less than 0.1
Alkali Na_2 (%)	0.4-1.0
Tricalcium silicate C_3S (%)	40-60
Dicalcium silicate C_2S (%)	12.5-30

*values according to manufacturer

Table 4.16 Other properties of PFA*

Properties	Typical value
Compacted bulk density (kg/m^3)	1300-1500
Surface area (m^2/kg)	300-400
Specific gravity (oven dry)	2.0-2.4
Specific heat capacity ($J\ kg^{-1}k^{-1}$)	0.7-0.8
Loss on ignition LOI (%)	6-10
Silicon oxide SiO_2 (%)	38-52
Aluminium oxide Al_2O_3 (%)	20-40
Iron oxide Fe_2O_3 (%)	6-16
Calcium oxide CaO (%)	1.8-10

*values according to manufacturer

Chemical compositions of other cements, PFA and RGD were collected from literature and listed alongside the counterparts used in this study as shown in Appendix 3 Table A.3.1 for comparative purposes.

When comparisons are made between the chemical composition of the author's RGD, PFA, and cement, it can be seen that RGD contains high silica and alumina content just like PFA. This on the other hand suggests a potential pozzolanic reaction and therefore a cementitious nature. Chemical requirements for fly ash as per BS EN 450-1:2005+A1:2007 dictate that the content of reactive silicon dioxide in fly ash shall not be less than 25%. This limit is far below that found in RGD (61.4%), but the reactivity is more important; materials could have high silicon dioxide content but be less reactive. Another limitation is that the sum of ferric oxide (Fe_2O_3) + alumina (Al_2O_3) + silica (SiO_2) must be at least 70% in PFA for possible use in concrete. The sum of these oxides is more than 81% in RGD, satisfying this requirement, which leads to the belief that RGD may be useful (like PFA) for concrete mixes.

(c) Scanning electron microscopy of cement, PFA, and RGD samples

The scanning electron microscope (SEM) is an instrument that produces a magnified image by using electrons instead of light. A beam of electrons is produced at the top of the microscope by an electron gun. This beam follows a vertical path through the sample chamber, which is under vacuum. The beam travels through electromagnetic fields and lenses, which focus the beam down toward the sample. Once the beam hits the sample, electrons and X-rays are ejected from the sample. Detectors collect these back-scattered electrons and X-rays for energy dispersive analysis (EDXA). They are converted into a signal that is displayed on screen and captured for storage and analysis.

A modern trend in electron microscopy is to fit X-ray analysis equipment as a bolt-on accessory. Bombarding a specimen with electrons causes X-rays of characteristic wavelengths and energies to be emitted from the spot where the beam strikes the specimen. Computer analysis of the wavelength or energy spectra makes it possible to measure accurately the nature and quantity of different elements in the material. The technique is of great value in materials science, particularly because an area as small as 1 square micron can be analysed with precision. This technology can be used to study the morphology, grain texture and size of particles and the material's features may be compared.

(i) SEM of cement

Figure 4.10 shows an SEM micrograph of a cement specimen and the energy spectrum (Fig. 4.11) of the characteristic X-rays emitted therefrom.

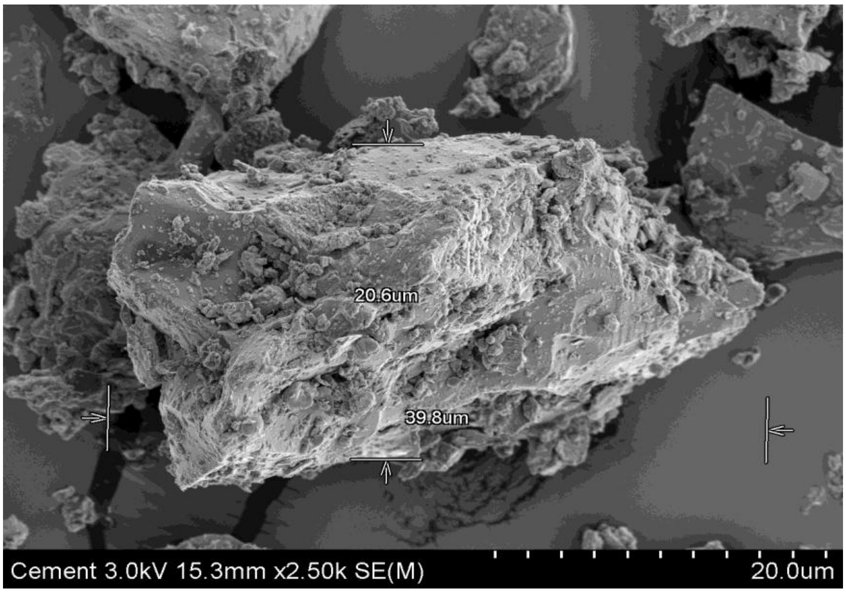


Fig. 4.10 SEM image of cement specimen

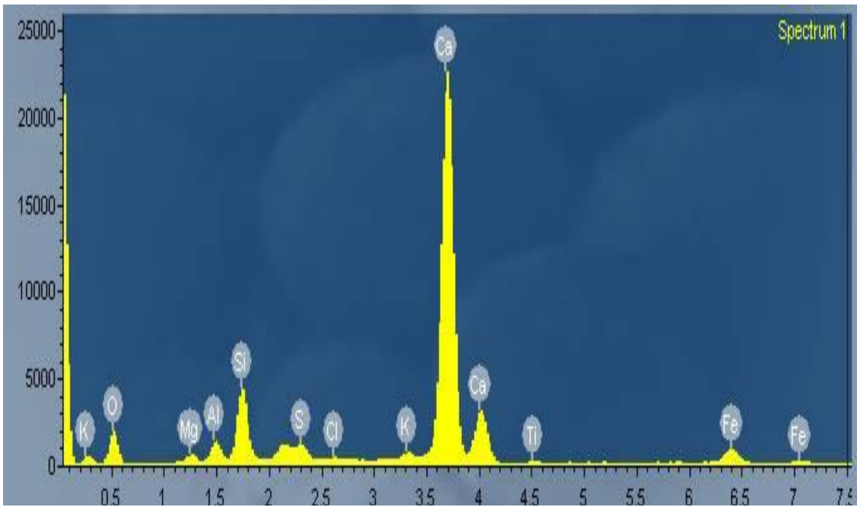


Fig. 4.11 Spectrum of the energy of characteristic X-rays emitted from the cement specimen

The vertical Y-axis shows the intensity or the counts; counts on spectrum are derived from the acquisition rate (operator variable); a higher number of counts mean better spectra. The peak lying on the Y-axis (at *c.* 0 keV) is known as the noise peak which shows that the system is working and can be removed if needed. The X-axis is the energy default 0-20 keV; this scale can be expanded as required to show the spectrum clearly.

Depending on which element is present, different lines of excitation or energy can be produced resulting in *e.g.* Ca K_{α} or Ca K_{β} transitions and that is the reason two Ca peaks appeared. Several sizes were shown on the images to show the size differentials. The machine used to produce the spectra is capable of element detection down to boron at atomic number Z = 5.

(ii) SEM of PFA

Figure 4.12 shows an SEM micrograph of the PFA specimen and the energy spectrum (Fig. 4.13) of the characteristic X-rays emitted therefrom.

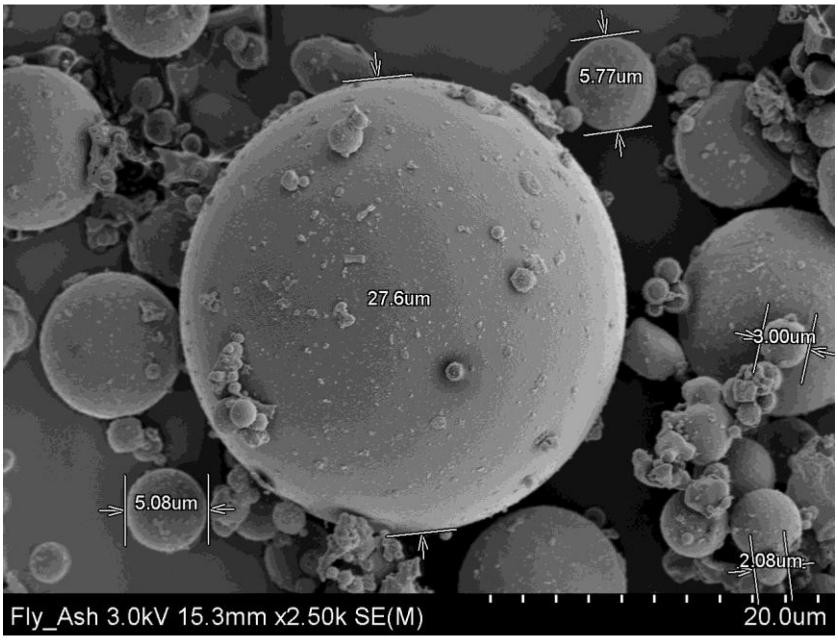


Fig. 4.12 SEM image of PFA specimen

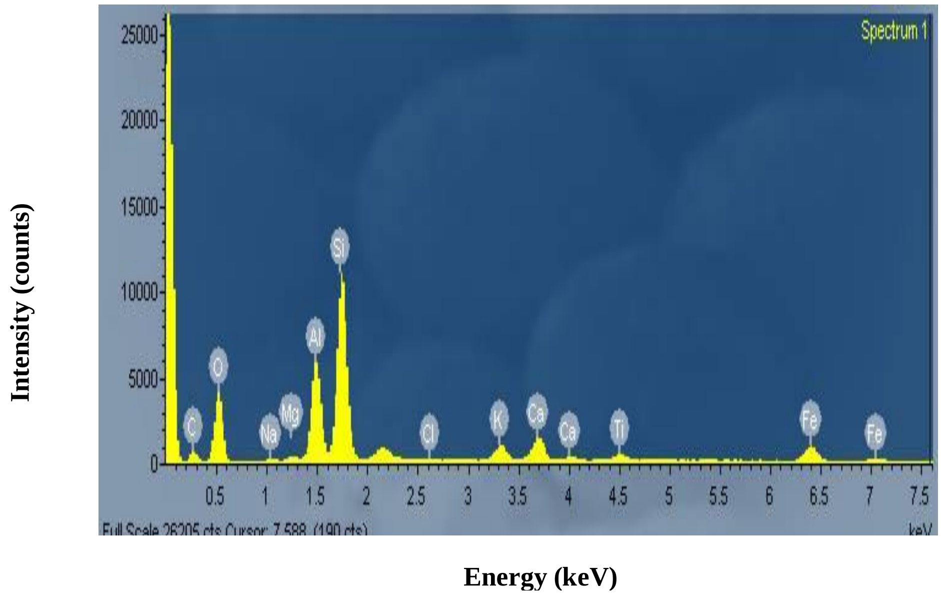


Fig. 4.13 Spectrum of the energy of characteristic X-rays emitted from the PFA specimen

(iii) SEM of RGD

Figure 4.14 shows an SEM micrograph of the RGD specimen and the energy spectrum (Fig. 4.15) of the characteristic X-rays emitted therefrom.

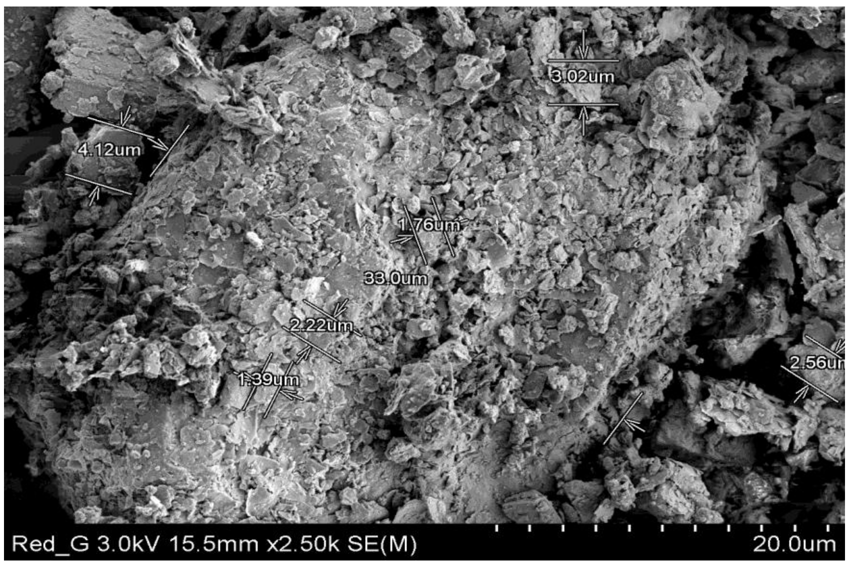


Fig. 4.14 SEM image of RGD specimen

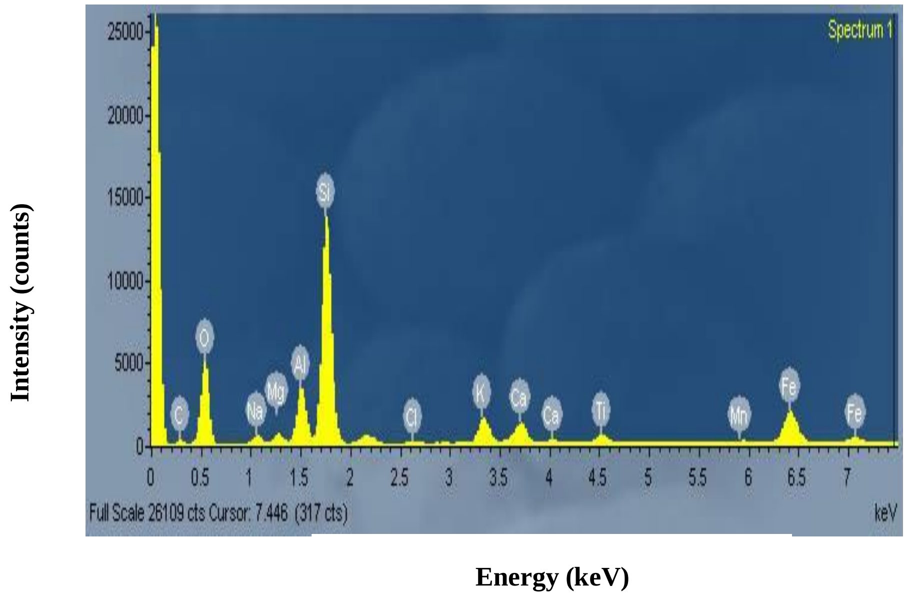


Fig. 4.15 Spectrum of the energy of characteristic X-rays emitted from the RGD specimen

The shape, texture and morphology of particles can be evaluated from the SEM images, while the concentration of different elements can be identified from the energy spectra by the X-ray peaks. The multiple peaks for individual elements are due to energy band gap variations. The images showed the shape was angular, rough and firm for cement particles; in contrast, the PFA particles were spherical, smooth and spongy. RGD particles were noticeably bigger, more angular and rougher. Images also showed that cement particles were relatively more porous than those of RGD and PFA. The sizes of particles were variable for the three materials; the RGD image showed more fragmentation. The spectra showed a similarity of elements and their concentration in both PFA and RGD specimens, particularly in silica and alumina content. Cement elements were different; the highest concentration was for Ca.

There are four chief minerals present in a Portland cement grain: tricalcium silicate (Ca_3SiO_5), dicalcium silicate (Ca_2SiO_4), tricalcium aluminate ($Ca_3Al_2O_6$) and calcium aluminoferrite ($Ca_4Al_nFe_{2-n}O_7$). The formula of each of these minerals is usually reduced down into the basic calcium Ca, silicon Si, aluminum Al and iron Fe oxides (CNX 2008 http://cnx.org/content/m16445/latest/). Relating the SEM micrographs of each material to the chemical composition (Tables 4.14 to 4.16) an agreement can be observed; for instance the highest count in cement is Ca, accounting for 65.1% similarly the highest concentration appeared in the SEM EDXA data. Ca is low in PFA (8.8%) and RGD (3.7%) and also appeared low in SEM EDXA data. Silcon content is high in

PFA (57%) and RGD (61.4%) samples and moderate in cement (21.8%); again there is agreement between the two types of analysis. Similar trends can be seen in Al content and other components of the three materials.

4.6 DESIGN OF CONCRETE MIXES

There is no single standard design method used for mix proportioning and many academic institutions and concreting companies worldwide have developed their own. The selection of concrete components is based on the intended use of concrete, however trial mixes are always strongly recommended. In the UK, the Building Research Establishment (BRE) Mix Design Method 1992, recommended by the UK Department of the Environment, was in use for a long time and proved to be a powerful method and ideal for locally-produced concrete materials in Scotland, and therefore, will be used to design all standard mixes of NAC and RAC mixes throughout this study. This method will be considered in the new European mix design, expected to be published in 2010.

4.7 SELECTION OF SP TYPE

(a) General

In this part of the study, cement and PFA at different levels will be used as a binder material and an SP will be used to guarantee adequate workability at low w/c ratio. Admixtures react differently with different cements (Neville 2003). Several studies (see Chapter 3 for more details) have pointed out that the SP should be compatible with the particular binder. The optimum amount of SP should be determined in advance before mass production of concrete. An under- or over-dose could lead to undesirable concrete properties such as stiff concrete, rapid slump loss; segregation, honeycombed structure, and generally low quality concrete (Neville 2003).

As stated earlier, the SP used in this study will be selected from three superplasticizers on the basis of their compatibility with the binder materials (cement and PFA) and the concrete's performance. The three SPs tested here are: Structuro 11180 compliant with EN BS EN 934-2: 2005, BS 5075-3: 1985; Structuro 115 compliant with BS EN 934-2: 2005; and Complast M1 compliant with BS 5075-3: 1985. These SP will be indicated from now on as SP A, SP B, and SP C respectively. Types A and B are high early strength new generation polycarboxylate polymers, and type C is a high early strength chloride-free SP based on sulphonated melamine polymers.

At this stage, the process of selecting the best SP type and dose on the basis of the setting times, water demand of the binder paste and the loss of workability of concrete over time will be detailed in this chapter; while selection on the basis of concrete performance as evaluated by its fresh and hardened states will be dealt with in subsequent chapters.

(b) Setting times of the binder paste

Instead of cement, combinations of cement and PFA at different levels will be used as a binding material. To examine the compatibility of each SP with the binder (cement and PFA), the initial and final setting times of the binder are tested.

The setting time of the paste is the transition from a fluid to a plastic state. For cement, depending on its type, this can occur in less than one hour or could take up to 24 hours. The initial setting time (IST) of cement paste is defined as the time by which the paste has gained enough rigidity to no longer be in a fluid state. The final setting time (FST) is the time by which rigidity has increased to a point that the paste becomes a solid of very low strength (Neville 1995). The standard normal consistency (NC) of the paste is the ratio of water added to the mix to the amount of binder material. The water added for NC is the exact amount of water that produces the standard plunger penetration of the Vicat consistency apparatus *i.e.* 5 ± 1 mm.

The Vicat method has been based on shearing cement paste with a needle and on the idea that stiffening during the set induces a gradual increase in resistance to shearing. Although the Vicat test's application was generalised in the 19th century, the test remains today the most widely used test by cement manufacturers and is the subject of multiple standards (BS EN 196: Part 3: 2005; NF EN 196-3; ASTM C191-93; AASHTO T 131) around the world (Sofiane 2006).

Cement pastes produced with different cement types and superplasticizers, with equal and dissimilar dosages, produced different initial and final setting times (Agarwal *et al.* 2000). This phenomenon however, was indicated by several other studies, and therefore, when materials such as PFA are mixed with cement to form a new binder paste rather than only cement, properties including setting times and the rate of slump loss could be contradictory to those of the cement paste. Different levels of these materials in the mix could yield different results. Moreover, adding an SP to this new binder could lead to different setting times compared with the equivalent cement mix or the binder without SP. It has been reported (Illston & Domone 2001) that when 30% fly ash was used to replace cement in a mix, the setting times were increased by up to 2 hours, thus resulting in a reduced rate of workability loss.

In this part of the study, PFA was substituted for the cement at percentages of 0, 25, 50, and 75%. Paste mixes were produced using the different types of SP at the levels given in Table 4.17. NC, IST, and FST for each mix were obtained using the Vicat apparatus in accordance with BS EN 196: Part 3: 2005. The aim is to test the compatibility of SP (A, B, C) with the binder and to select the most suitable SP, the one that produces appropriate setting times and workability of mixes, in other words the SP most compatible with the binder.

Standard NC, IST, and FST were carried out in accordance with BS EN 196-3: 2005, for different paste mixes produced with different percentages of PFA and SP. Superplasticizers were added to the mixture taking into account manufacturer recommendations from which suitable dosages of superplasticizers are 0.6 - 1% for SP A, 0.4 - 0.8% for SP B and 2 - 3% for SP C, therefore, average levels of 0.8, 0.6, and 2.5% of the binder for SP A, B, and C respectively were selected. Results are summarised in Table 4.17.

Table 4.17 Normal consistency and setting times of binding paste with different percentages of PFA and different types of SP

PFA replacement level (%)	SP type	SP (%)	Normal consistency, water demand and setting times of the binder paste			
			NC (%)	Water demand (%)	IST (hr)	FST (hr)
0	-	0	28.85	-	2.25	4.10
0	A	0.8	22.1	-23	6.75	9.42
25	A	0.8	23.14	-19	8.73	11.0
50	A	0.8	26.26	-9	8.18	10.2
75	A	0.8	28.57	-1	7.45	10.2
100	-	0	35.43	+23	4.35	5.45
0	-	0	28.85	-	2.25	4.10
0	B	0.6	21.71	-25	3.22	4.20
25	B	0.6	24.57	-15	4.10	5.43
50	B	0.6	28.57	-1	4.27	5.22
75	B	0.6	30	+4	3.36	4.35
0	-	0	28.85	-	2.25	4.10
0	C	2.5	22.85	-21	4.29	5.35
25	C	2.5	23.48	-19	6.10	7.32
50	C	2.5	25.20	-13	8.08	10.1
75	C	2.5	28.57	-1	11.2	13.1

Negative values indicate water reduction in producing consistent paste.
Results listed above are average values of three measurements.

Data in Table 4.17 are plotted in Figs 4.16 and 4.17 to show the influence of SP (A, B and C) on the IST and FST respectively; each bar is the average of three samples.

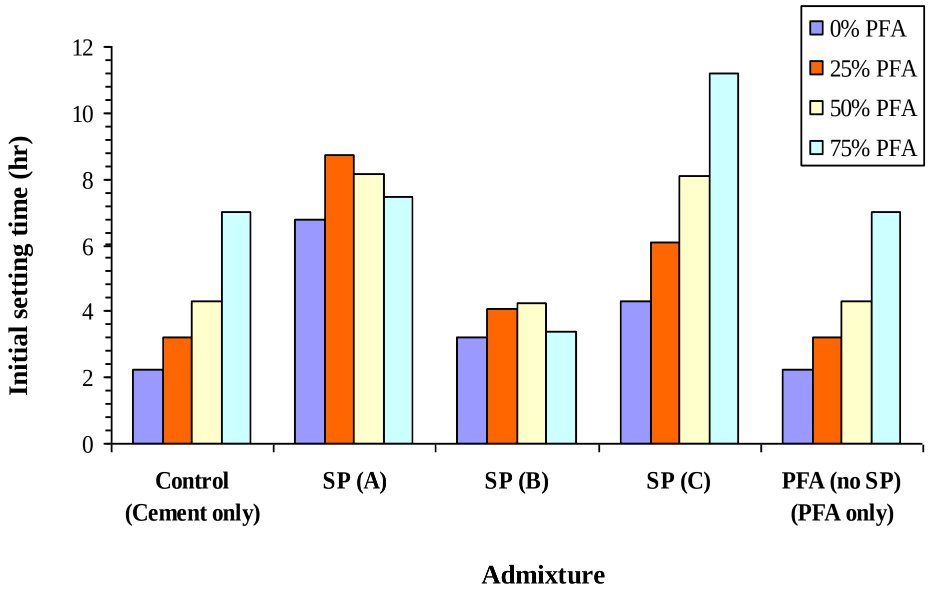


Fig. 4.16 Influence of SP type on the binder's IST

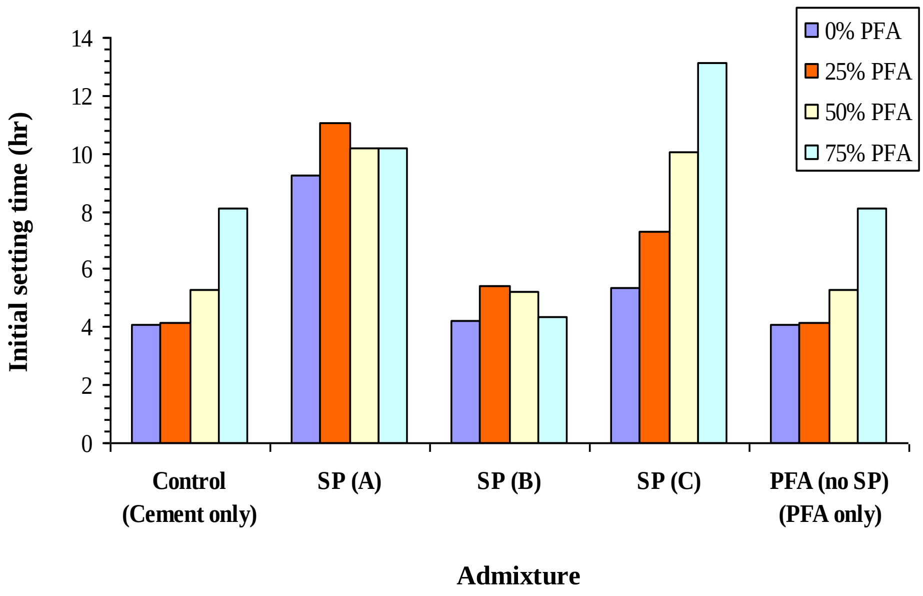


Fig. 4.17 Influence of SP type on the binder's FST

Results in Figs 4.16 and 4.17 showed that the incorporation of PFA at all levels tested, can increase the setting times of the binding paste made with SP types A and C as compared to control mix, although at different rates depending upon the percentage of

PFA. The extension was marginal when SP type B was used. In mixes with only PFA (without SP), the times were marginally extended for replacement levels below 50% but significantly increased for higher PFA volume.

Figs 4.16 and 4.17 also revealed that when SP types A and B were used, the setting times were extended almost equally for all PFA levels, but the case was different when SP type C was used, the setting times were significantly influenced by the level of PFA in the mix; setting times increased with the PFA content. SP type B achieved the shortest setting times; therefore, the use of SP B could lead to increased slump loss and minimum time for concrete transportation and placement. Setting times were normalized with respect to cement paste (control mix) and re-plotted in Figs 4.18 and 4.19. Normalization gives a good idea of the relative variation of setting times.

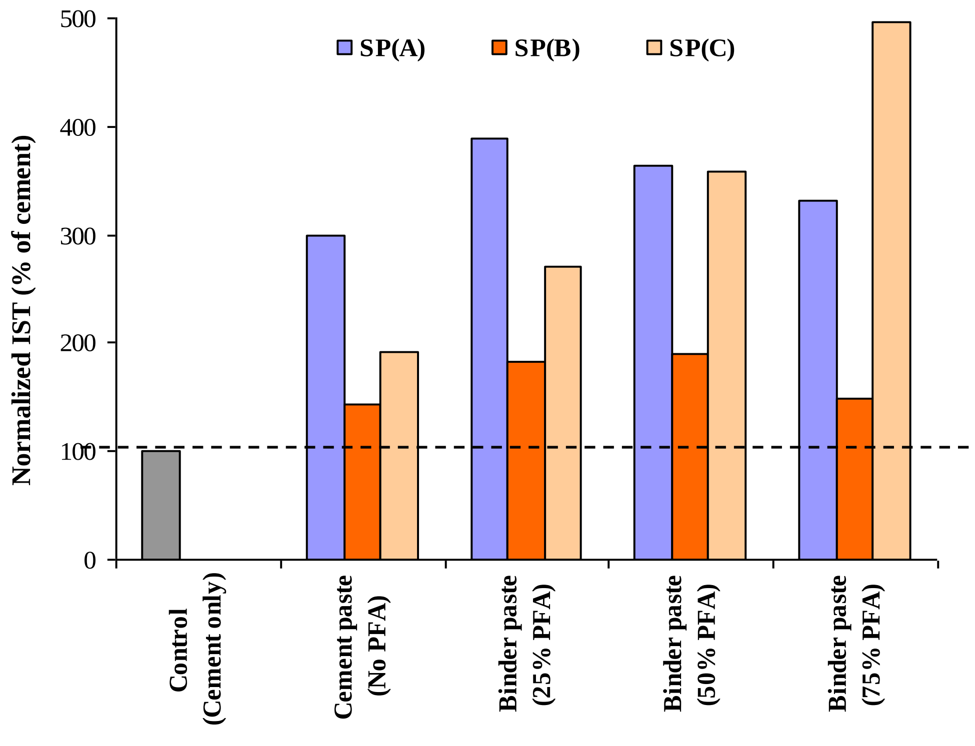


Fig. 4.18 IST of mixes at different level of PFA and SP with respect to standard mix

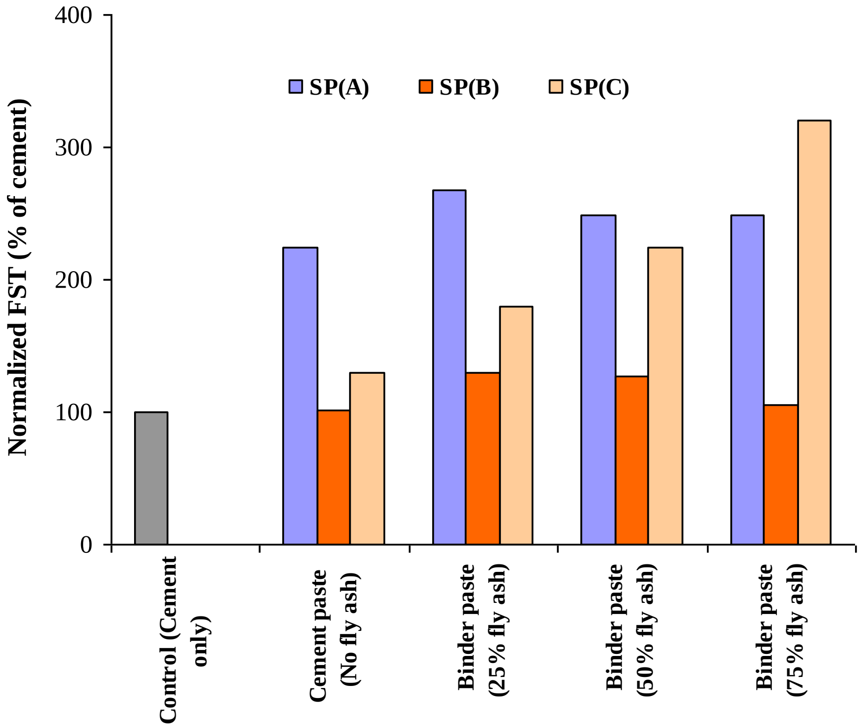


Fig. 4.19 FST of mixes at different level of PFA and SP with respect to standard mix

Based on the test results and preceding discussion, SP A is the most suitable option to be used for cement and PFA mixes, in other words it is the most compatible chemical admixture out of the three tested here.

(c) Influence of SP and PFA on water demand of the paste

The influence of SP on water demand of the binder paste (cement and PFA) was also studied. The aim is to find the ideal percentage of PFA and select the best SP; the SP was already selected but the relationship between the PFA levels in the binder of the mixes made with different SP led the author to double-check the setting time conclusion. Results are displayed in Fig. 4.20. In mixes with only PFA replacing cement (without SP), water demand was increased from 0% for no PFA mixes to 20% for the mix containing 75% PFA.

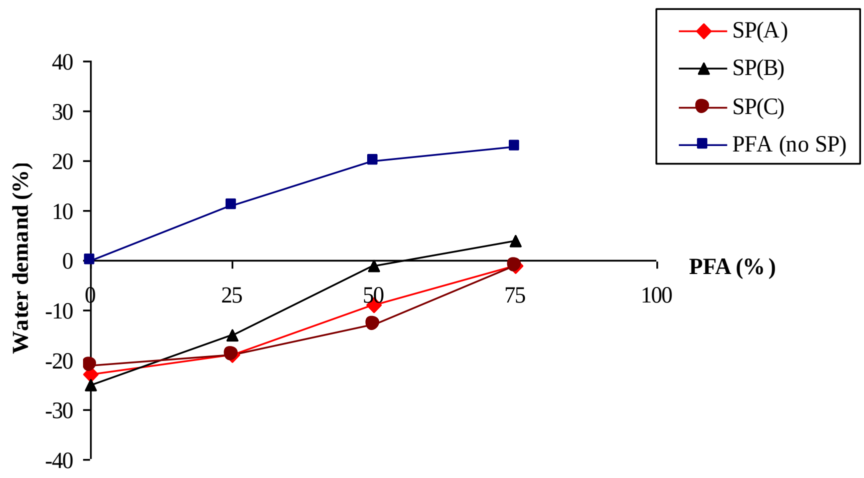


Fig. 4.20 Influence of SP type and PFA on water demand of binder paste

For cement paste (0% PFA), the water demand required to make a consistent mix decreased with all types of SP, the reductions in water requirement were 21, 23, and 25% when SP C, SP A, and SP B were used respectively. Water demand for mixes made with SP and PFA, generally increased as the level of PFA in the mix increased. This is probably due to the increased surface area of the binder, as the PFA used here is finer than cement (confirmed by wet sieve and SEM analysis as detailed in this chapter).

From Fig. 4.20, for the case of no extra water intake, PFA contents of up to 50% of the binder can be used when SP B is employed, and up to 75% PFA when SP A and SP C are used. It is apparent the superplasticizers were capable of keeping water demand to a minimum for higher PFA content. Therefore, as far as water demand is concerned, the PFA content of the binder material should be limited to less than 50% when SP B is used. On the other hand, the level could be extended to 75% when SP types A or C are used.

Again the results support the earlier selection of SP type A for further research. However, the behaviour of the mix could be different when these admixtures *i.e.* the PFA and superplasticizers are added to other concrete components, rather than the binder paste, to make concrete; the influence may include both fresh and hardened properties. The most important perhaps are the loss of slump over time and the mechanical performance of hardened concrete.

(d) Slump loss of RAC mixes

Another important factor influencing the selection of the appropriate type and dosage of SP to be used with a binder is the loss of workability with time. Slump loss is directly related to the depletion of free water in fresh concrete (Şakir 2005). The simplest, and crudest, way to measure the workability is the standard slump test to BS 1881: Part 102: 1983. The Vebe test is also used; the Vebe time is defined as the time taken to completely remould a sample of concrete from a slump test; times between 1 and 25 s are usual, with higher values indicating decreased workability (Illiston & Domone 2001).

Chemical admixtures in concrete can cause rapid slump loss over time to ordinary cement mixes (Neville 1995). The increased fluidity of the mortar is attributed to the adsorption of SP on the surface of cement particles which hinders hydration, and the slump loss occurs as a result of the resumption of hydration (Chandra & Björnström 2002). The mechanism of slump loss is outwith the scope of this study; however it involves so many physical and chemical interpretations and has been a subject of many previous studies including tests on superplasticized mixes. Slump loss should really be assessed by trials on the material at hand and under relevant site conditions.

Several studies (Collepardi 1998; Chandra & Björnström 2002; Şakir 2005) have reported that slump loss is higher in superplasticized concrete with respect to the corresponding plain mix at a given initial slump. The loss is more pronounced when traditional sulphonated naphthalene formaldehyde base admixtures are used. Lower w/c ratio of superplasticized concrete mixes is likely to cause significant slump loss as the available amount of free water is lost by reaction with cement and through evaporation during transport (Liu *et al.* 2000).

In view of the aforesaid reasons, the effect of superplasticizers on slump loss of concrete mixes incorporating PFA was examined. The aim was to investigate the influence of SP on the workability of concrete mixes, as measured by the rate of slump loss. In addition to the workability monitoring, the test will help to evaluate the selection of the SP already made based on the setting times and water demand data analysis.

PFA can be used either to partially replace cement or fine aggregate (see Chapter 3 for more information); in this study it is intended to examine both options. Therefore, eight concrete mixes in which PFA partially substituted fine aggregate at percentages of 0, 25, 50 and 75% by weight using SP type A at constant rate of 0.8% of binder content were used. Four other mixes with constant 50% PFA replacement in which SP type B at 0.6% of binder content and SP C at 2.5% binder content was produced.

Mixes in this section were produced solely for the above mentioned aim, and therefore, a small laboratory mixer with a 0.011 m^3 tank was used to produce 0.0083 m^3 of concrete, quite enough for slump measurement. Standard NAC and RAC mixes for slump monitoring purposes were designed to have high slump (60-180 mm) to allow monitoring of the rate of slump loss over longer time (90 minutes). Lower initial slump can be lost quickly, and thus the monitoring process may be restricted. Slump and Vebe time were measured every 30 minutes over a period of 90 minutes in accordance with BS EN 12350-1: 2000 and BS EN 12350-2: 2000. Spreading diameter of the initial slumps was also measured. Mix proportions for slump monitoring purposes are shown in Table 4.18, while the complete mix design is shown in Appendix 2.

Table 4.18 Mix proportions of standard mix for slump monitoring purpose

Material	Mass used (kg/m^3)	
	NA	RA
Cement	445	445
Water	205	205
Coarse aggregate	1010	900
Fine aggregate	700	685
Wet density	2360	2325

Due to the difference in relative density and specific surface area of PFA and fine aggregate, the amounts in Table 4.19 are adjusted according to the following sample calculations:

Table 4.19 Adjusted mix proportions used for slump monitoring purpose

Material	PFA percentage (%)							
	0		25		50		75	
Aggregate type	NA	RA	NA	RA	NA	RA	NA	RA
Cement	445	445	445	445	445	445	445	445
Water	205	205	205	205	205	205	205	205
Coarse aggregate	1010	900	1010	900	1010	900	1010	900
Fine aggregate	700	685	525	514	350	342.5	175	171
PFA	0	0	175	171	350	342.5	525	514
PFA used	0	0	151	148	303	296	454	445

PFA used is the amount that produces an equal volume of the substituted fine aggregate

1 For 25% PFA replacement, the standard amount *i.e.* the mix design amount of fine aggregate was 700 kg/m^3.

2 PFA level was 25% and fine aggregate was 75%, therefore the amount of PFA is $0.25 \times 700 = 175$ kg/m^3, and the amount fine aggregate is $0.75 \times 700 = 525$ kg/m^3.

3 The relative density of fine aggregate and PFA were 2.6 and 2.25 respectively, therefore, the actual amounts of fine aggregate used were 525 kg/m^3 and for PFA $175 \times 2.25/2.6 = 151$ kg/m^3

Testing results showing the influence of SP and PFA on slump loss over time of NAC and RAC mixes are listed in Table 4.20.

Table 4.20 Influence of SP and PFA on slump loss over time of NAC and RAC mixes

Mix no	Mix	SP type	PFA (%)	Slump (mm), Vebe time (s) and slump loss after: 0 hr	½ hr	1 hr	1 ½ hr
M1	NAC-PFA0 Slump (mm) Vebe (s) Slump loss (%) (Control mix)	A	0	220 - -	90 5 59	2 8 99	0 8 100
M2	RAC-PFA0 Slump (mm) Vebe (s) Slump loss (%) (Control mix)	A	0	220 - -	75 4 66	0 9 100	0 - 100
M3	NAC-PFA25 Slump (mm) Vebe (s) Slump loss (%)	A	25	210 - -	107 6 50	135 7 65	0 14 100
M4	RAC-PFA25 Slump (mm) Vebe (s) Slump loss (%)	A	25	225 - -	55 3 76	37 6 84	26 10 89
M5	NAC-PFA50 Slump (mm) Vebe (s) Slump loss (%)	A	50	215 - -	106 5 51	50 7 77	35 8 84
M6	RAC-PFA50 Slump (mm) Vebe (s) Slump loss (%)	A	50	210 - -	30 8 85.7	0 15 100	0 - 100
M7	NAC-PFA75 Slump (mm) Vebe (sec) Slump loss (%)	A	75	220 - -	130 4 41	95 5 57	40 6 82
M8	RAC-PFA75 Slump (mm) Vebe (s) Slump loss (%)	A	75	220 - -	70 5 68	68 5 69	36 7 84
M9	NAC-PFA50 Slump (mm) Vebe (s) Slump loss (%)	B	50	215 - -	85 4 60.4	25 7 88.3	0 15 100
M10	NAC-PFA50 Slump (mm) Vebe (s) Slump loss (%)	B	50	220 - -	15 7 93	3 10 98	0 14 100
M11	NAC-PFA50 Slump (mm) Vebe (s) Slump loss (%)	C	50	220 - -	30 6 86	30 6 86	28 7 87
M12	RAC-PFA50 Slump (mm) Vebe (s) Slump loss (%)	C	50	215 - -	40 6 81	27 7 87	6 9 97

SP were added in accordance with manufacturer recommendations at constant values of 0.8, 0.6 and 2.5% of the binder content for SP types A, B, and C respectively.

Due to the high mix design slump of the standard mixes (60-180 mm) and the addition of the SP, the initial slump of all mixes *i.e.* slump at t = 0 were collapses resulting in high measured slumps of 215 ± 5 mm, and flow spread diameters of 460 ± 10 mm for NAC mixes and 440 ± 10 mm for RAC mixes depending on the mix's PFA content. Table 4.20 shows that at least 80% of slumps were lost within 30 minutes. For control mixes (M1 and M2), the rate of slump loss was almost the same for NAC and RAC mixes, about 60% of slump was lost within 30 minutes and nearly 100% after 90 minutes. To show the effect of SP type on slump loss, results of slump loss monitoring data at 30 minutes in Table 4.20 are plotted in Fig. 4.21.

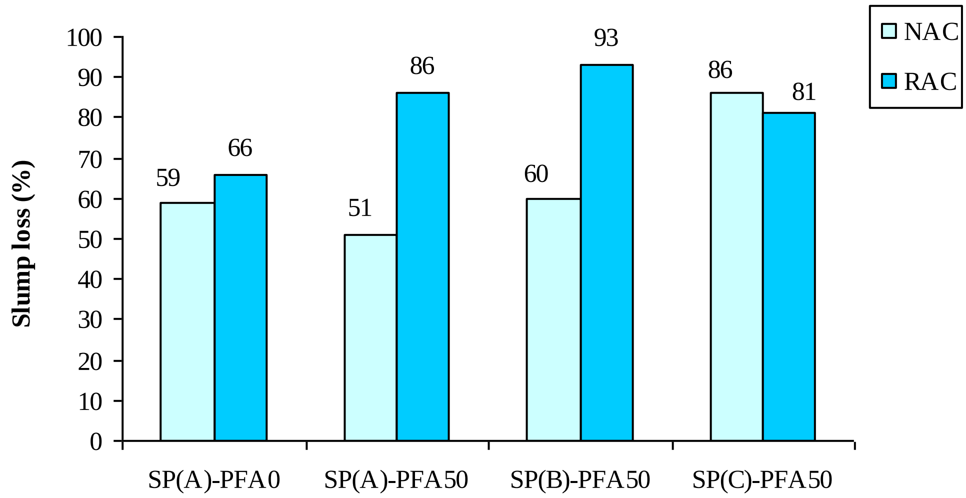


Fig. 4.21 Influence of SP type on slump loss at 30 minutes for mixes with 50% PFA replacement

Fig. 4.21 shows that SP A produces the least slump loss for concrete mixes; about 50% and 86% loss at 30 minutes for NAC and RAC respectively. Results support the selection of SP type A for further work. Results showed, in general, that the rate of slump loss for RAC mixes was slightly higher than that for NAC despite the type of SP; that is probably because of the higher porosity of the RA and the presence of old cement paste which tended to absorb more water.

To show the effect of PFA content on the rate of slump loss, data in Table 4.20 (M1 to M8), in which SP type A was used, were presented in graphical form as shown in Figs 4.22 and 4.23 for NAC and RAC mixes respectively.

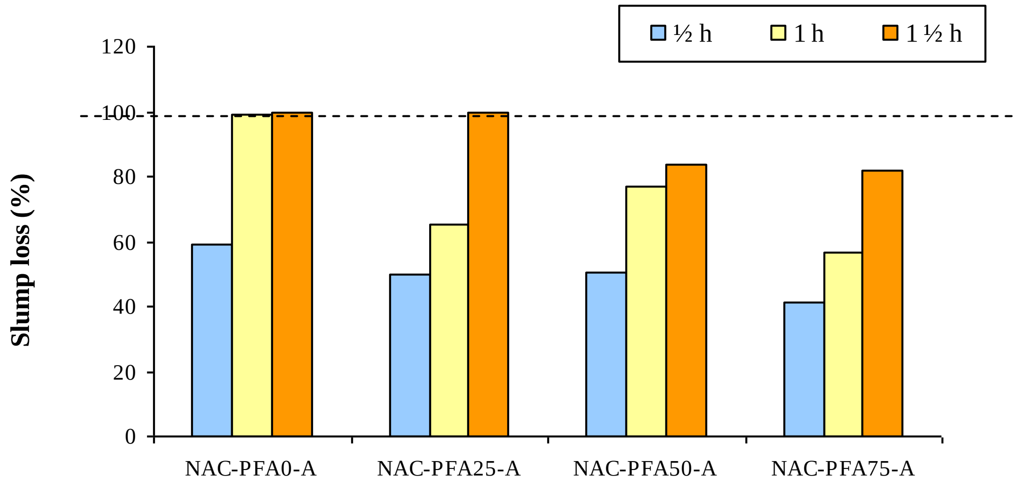


Fig. 4.22 Influence of PFA content on the rate of slump loss of NAC made with SP A

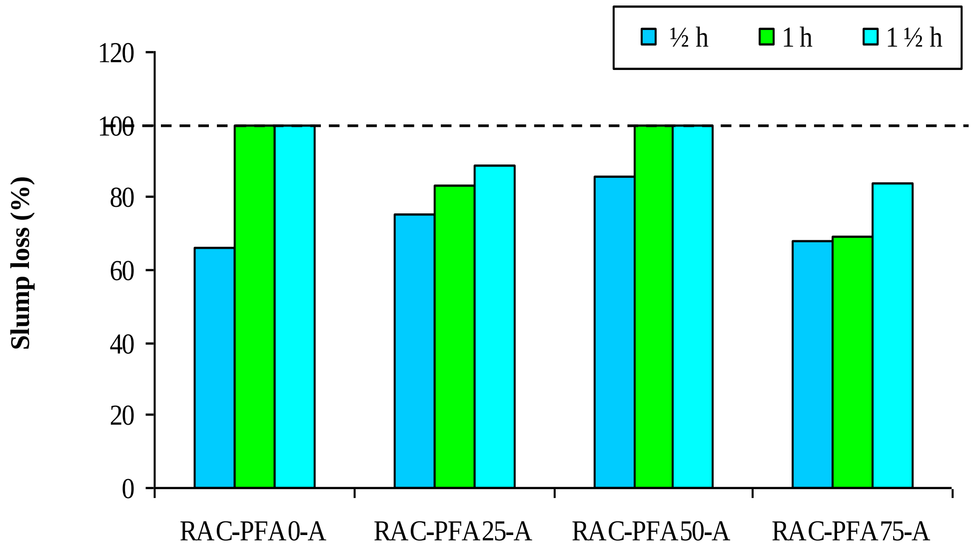


Fig. 4.23 Influence of PFA content on the rate of slump loss of RAC made with SP A

Table 4.20 and Figs 4.21 to 4.23 show the rate of slump loss of concrete containing PFA at different levels; all PFA levels tested can marginally reduce slump loss to a different extent. Therefore it helps to keep the concrete more workable, particularly, during production and handling. However, regardless of aggregate type, PFA generally has little effect on mitigating the problem of rapid slump loss over time, a phenomenon known to be inherent to superplasticized concrete. For equal PFA content of 50%, as far as slump loss is concerned, all SP types tested showed similar performance and this again supports the selection of SP type A for further work.

In contrast to the case when PFA was tested for setting times of the binder paste, in which it was revealed that the incorporation of PFA increased the setting time of the binding paste with all types of SP used, particularly types A and C, different behaviour was observed in concrete mixes. Therefore, reliance on results obtained from the testing of binder pastes involving SP and PFA could result in misleading conclusions; this however could be the case with fresh and hardened properties.

4.8 SELECTION OF APPROPRIATE WATER REDUCTION FOR CERTAIN SP

The manufacturer recommended SP dosage, usually, works properly with the materials used on a particular job (Agarwal *et al.* 2000). However, due to the variability of material and regional differences, the suitability of a given SP and its dosage should be based on trial mixes of the materials at hand.

Past experience with superplasticizers has clearly established their influence on the workability of concrete. When an SP is introduced to a concrete mixture, slump will increase, depending on the amount of SP. Water reducing superplasticizers are used to maintain the workability of concrete at low w/c ratios, and therefore water should be reduced when cement is kept constant to reduce the w/c ratio. There should be a limit to which water can be reduced as the SP dose is increased. Several studies (Agarwal *et al.* 2000; Chandra & Björnström 2002; Papayianni *et al.* 2005; Tayyeb *et al.* 2009) have shown, directly or implicitly, that inappropriate doses of SP and water reduction (WR) could lead to undesirable properties of fresh and hardened concrete, in both short- and long-terms.

In this study, the type of SP was selected, *i.e.* SP type A; at this stage the appropriate dose and WR should therefore be selected. Now, the question arises is what are the best combinations of SP dose and WR for the materials at hand? The trials presented here are an attempt to answer this question. A concrete mix was designed to have a characteristic strength of 50 N/mm^2 after 28 days with a slump of 10-30 mm. Mix proportions of standard mixes used for the selection of appropriate WR for a certain SP are shown in Table 4.21; while the complete mix design is given in Appendix 2.

Table 4.21 Mix proportions of standard mixes used for the selection of WR

Material	Mass used (kg/m^3)	
	NA	RA
Cement	425	425
Water	170	170
Coarse aggregate	1073	1220
Fine aggregate	660	550
Wet density	2400	2365

The SP dosage will be determined on the basis of trial concrete mixes. Results will help to obtain the appropriate dosage of SP and the corresponding WR which should be proportioned in concrete mixes which will be produced for further research on these materials.

The procedure followed here is based on a practical trial and error approach; the observations made as the testing proceeds help to decrease the number of trials required. In these trials, water was reduced by percentages of 0, 15, 20, and 25% of the initial mix design water while SP is introduced incrementally and the standard slump measured for every increment. The water reduction percentages of 0, 15, 20, and 25% were selected as preliminary trials with larger WR of 35 and 30% producing very stiff concrete, and needing large SP dosages to achieve the design slump (10-30 mm).

It has been observed that slump is very much time-dependent; for instance, a mix will produce 80 mm initial slump (slump at t = 0) while a similar mix produces 60 mm slump when measured 5 minutes later. The slump was also affected by the addition time of SP; the slump measured for a mix in which SP was introduced after 2 minutes was not the same, or at least not comparable to, the slump of the same mix in which SP was introduced after 1 minute or 3 minutes. Therefore, it was concluded that it is necessary to harmonize the addition time as well as the time when slump is measured for all trial mixes, that is to standardize the slump measurement of these mixes.

Trial mixes were produced with 0, 15, 20 and 25% WR from the standard mix so that the water to cement ratio (w/c) of the mix was reduced from 0.4 for the standard mix to 0.34, 0.32, and 0.3 respectively. The SP is introduced to all mixes at the same time during mixing and the slump measured at equal times. The relationship between SP percentage (SP %) and the measured slumps is shown on Figs 4.24 to 4.27. Results from trial mixes are also summarised in Tables 4.22 to 4.25 inclusive for WR of 0, 15, 20, and 25% and SP percentages of 0.6, 0.8, 1.0, 1.2 and 1.4% of the binder content respectively.

The test procedure was:

- An amount of SP is taken in a small jar and weighed before mixing.
- A small random quantity of SP is added to the concrete mixture after about 4 minutes' mixing and the remaining quantity of SP is weighed again.
- At about 5 minutes, the first standard slump is measured.
- A second amount is added to the mixture, and the second slump is measured after around 10 minutes and the remaining quantity of SP is weighed
- The previous step is repeated three more times.

In every case, the amount of SP added to the mix is the difference between the amounts of SP in the jar before addition minus the amount thereafter. The cumulative quantity is the addition of the current amount and the amount already in the mixture. The SP percentage is the ratio of weight of SP to that of the binder.

(a) Slump of control mix (0%WR)

Results of slump measured at different percentages of SP at 0% WR were displayed in Table 4.22. A sample calculation is shown below.

Table 4.22 Slump for different SP percentage for control mix at 0% WR

SP in jar (g)	SP added (g)	Cumulative SP (g)	SP (%)	Slump (mm)	Time	
					min	sec
87.13	-	-	-	-	-	-
80.33	6.8	6.8	0.21	10	5	20
67.85	12.48	19.28	0.6	30	10	10
59.07	8.78	28.06	0.87	40	13	40
53.42	5.65	33.71	1.1	120	17	10
38.17	15.25	48.96	1.52	Collapsed	20	10

Sample of calculations:

-Mass of the binder for the trial mix was 3230 g (the mix design quantity was 425 kg/m^3 and the volume of this trial was about 0.0076 m^3).

-The first amount of SP added was 6.8 g.

-The second amount of SP added was 12.48 g.

-The cumulative amount thus was 6.8 + 12.48 = 19.28 g.

-The first percentage of SP (SP (%)) was 6.8/3230 × 100 = 0.21% and the second was 19.28/3230 × 100 = 0.6%.

-Other SP percentages were obtained in the same way.

Graphs of SP percentages against the measured slumps are plotted for each case of water reduction.

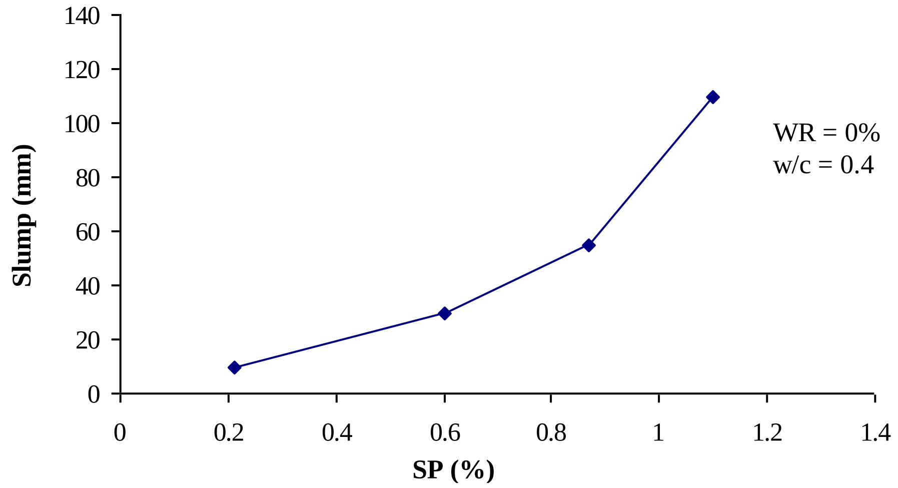


Fig. 4.24 Slump against SP relationship for control mix (0% WR)

(b) Slump measurements for 15%WR

Table 4.23 Slump for different SP percentage at 15% WR

SP in jar (g)	SP added (g)	Cumulative SP (g)	SP (%)	Slump (mm)	Time	
					min	sec
110.8	-	-	-	-	-	-
90.5	20.3	20.3	0.63	15	5	30
85.8	4.7	25	0.77	20	9	45
79	6.8	31.8	0.98	30	15	50
69.3	9.7	41.5	1.30	70	19	25

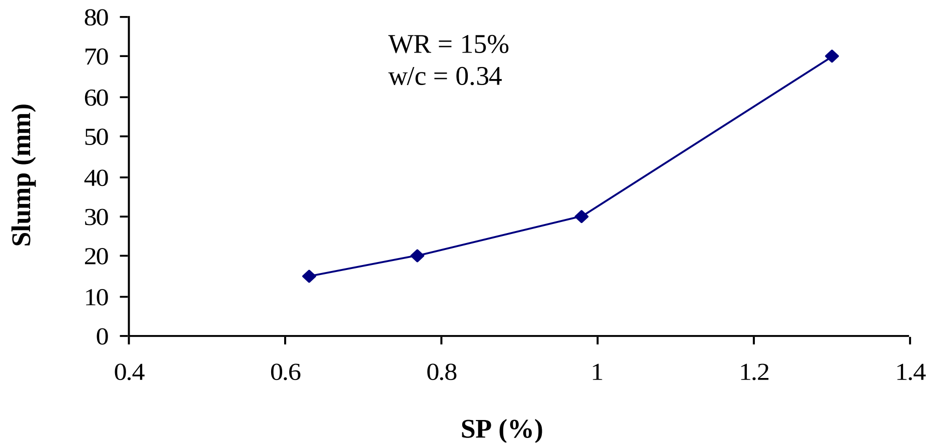


Fig. 4.25 Slump against SP relationship for a mix of 15% WR

(c) Slump measurements for 20%WR

Table 4.24 Slump for different SP percentage at 20% WR

SP in jar (g)	SP added (g)	Cumulative SP (g)	SP (%)	Slump (mm)	Time	
					min	sec
107.17	-	-	-	-	-	-
84.62	22.5	22.5	0.71	5	5	5
78.63	5.99	28.49	0.89	10	9	35
71.01	7.62	36.11	1.13	20	13	10
63.77	7.24	43.35	1.35	25	18	20
54.1	9.67	53.02	1.66	80	22	25

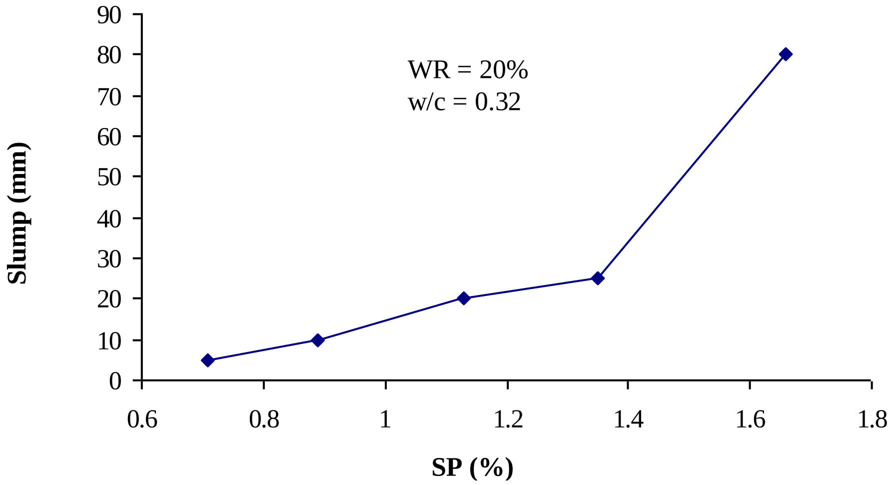


Fig. 4.26 Slump against SP relationship for a mix of 20% WR

(d) Slump measurements for 25%WR

Table 4.25 Slump for different SP percentage at 25% WR

SP in jar (g)	SP added (g)	Cumulative SP (g)	SP (%)	Slump (mm)	Time	
					min	sec
153.3	-	-	-	-	-	-
138.67	14.63	14.63	0.45	0	5	40
128.69	9.98	24.61	0.77	5	10	10
110.8	17.89	42.5	1.33	20	14	0
104.34	6.46	48.96	1.53	30	19	0
97.07	7.27	56.23	1.76	40	23	50

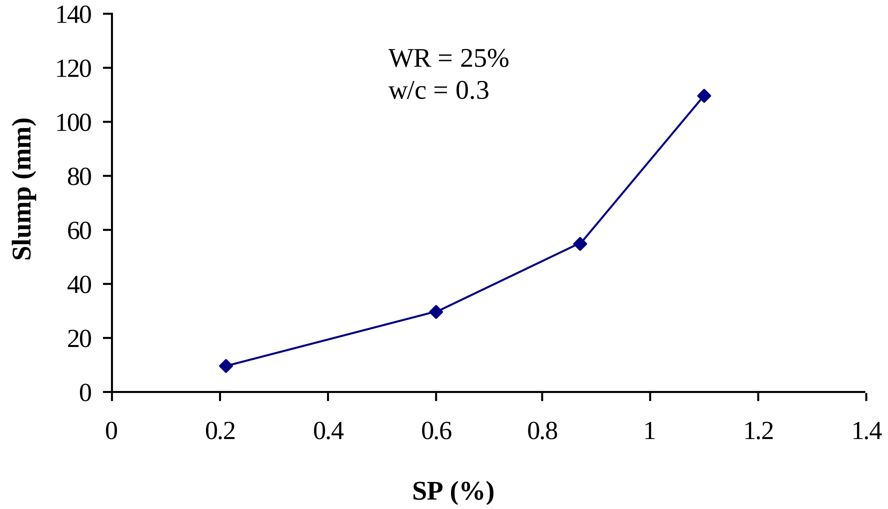


Fig. 4.27 Slump against SP relationship for a mix of 25% WR

(e) Relationship between water reduction and slump

To establish the relationship between WR and slump for a certain percentage of SP, the relationships between slump and SP, drawn above for different WR, can be employed. The objective is to determine the best combination of SP and WR for the materials which will be used for further investigations in this study. This however can be achieved by making use of the trend line (best-fit line) by which the amount of slump for different SP values can readily be read. Selecting the SP values as 0.6, 0.8, 1, 1.2 and 1.4%, the corresponding slump values can be obtained using the graphs above. The readings were then arranged in Table 4.26 from which another set of graphs were produced representing the relationship between WR and slump at different SP percentages. Results are displayed in Figs 4.28 to 4.32.

Table 4.26 Relationship between slump and WR of trial mixes at different SP percentages

WR (%)	Slump (mm) for the amount of SP of:				
	0.6%	0.8%	1.0%	1.2%	1.4%
0	40	60	80	100	120
15	8	20	40	55	75
20	0	5	18	30	50
25	0	5	20	20	30

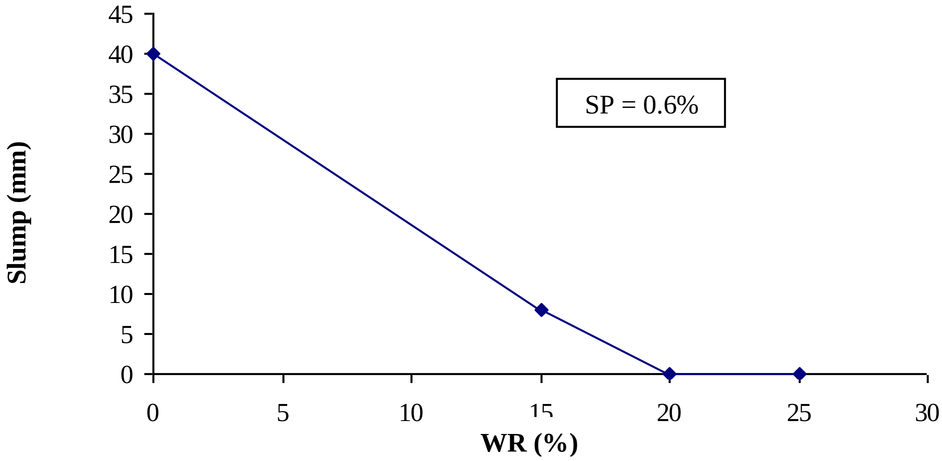


Fig. 4.28 Relationship between slump and WR for 0.6% SP

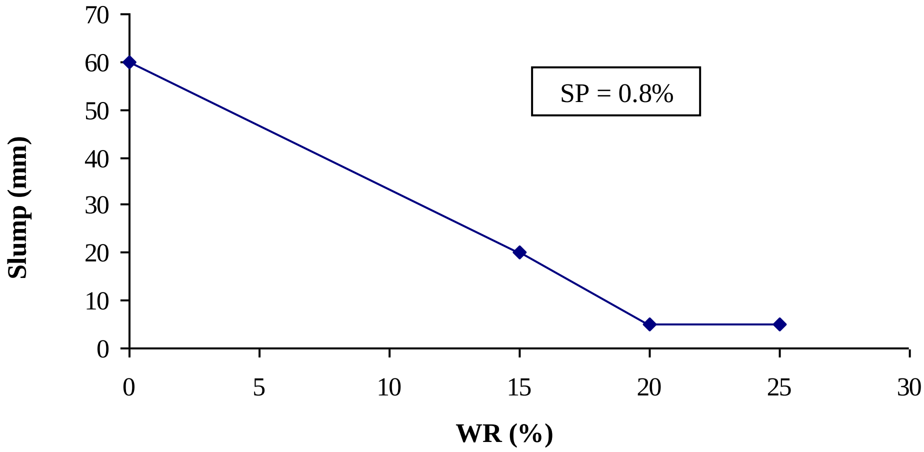


Fig. 4.29 Relationship between slump and WR for 0.8% SP

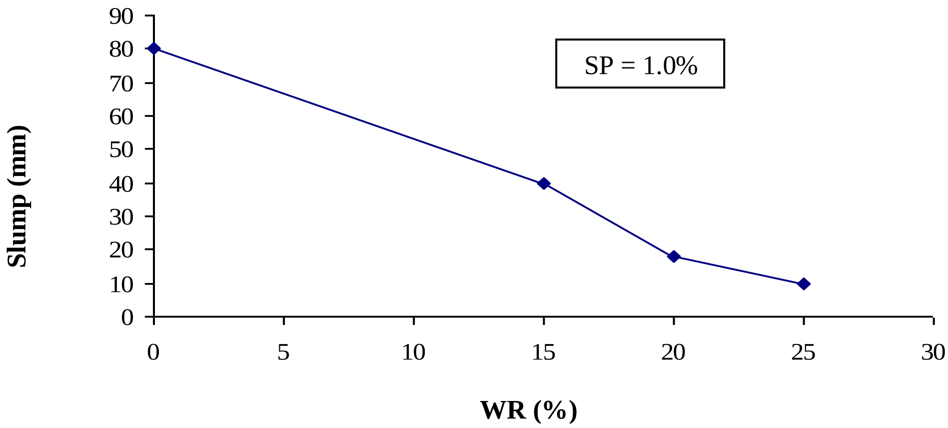


Fig. 4.30 Relationship between slump and WR for 1.0% SP

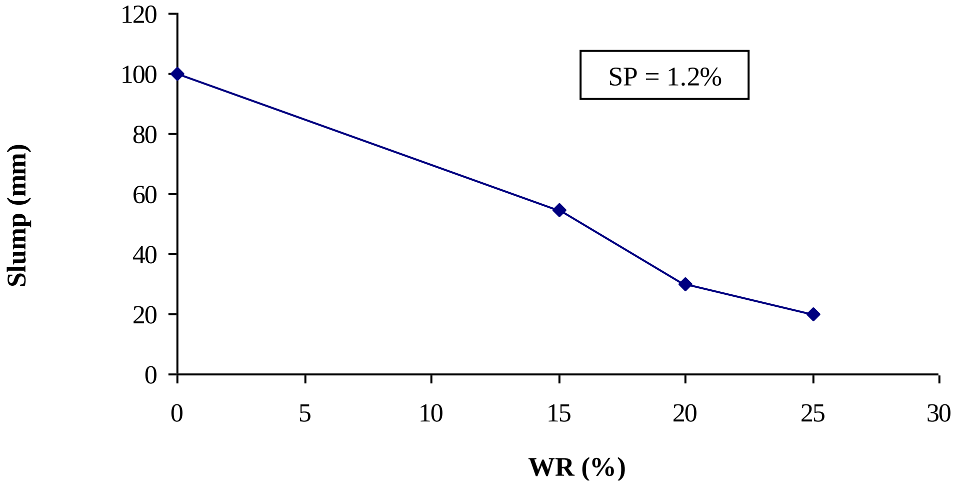


Fig. 4.31 Relationship between slump and WR for 1.2% SP

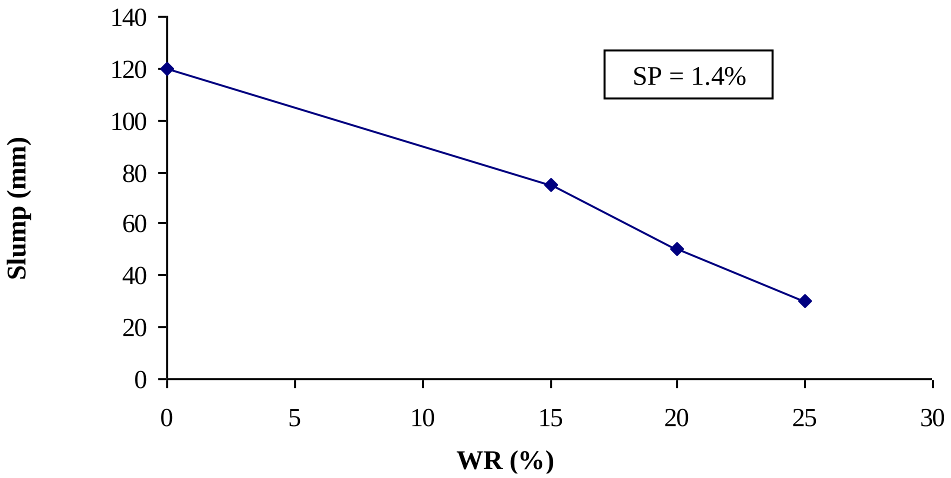


Fig. 4.32 Relationship between slump and WR for 1.4% SP

Graphs relating the WR and slump values were then used to obtain the level of WR for a given slump. The average value of the mix design slump was 20 mm (the mix was designed to have a slump of 10-30 mm). WR values for 0.6%, 0.8, 1.0, 1.2, and 1.4% SP were determined from the graphs above and listed in Table 4.27.

Table 4.27 The percentage of SP and the corresponding WR

SP (%)	WR (%)	Reference graph
0.6	12	Fig. 4.28
0.8	16	Fig. 4.29
1.0	22	Fig. 4.30
1.2	25	Fig. 4.31
1.4	29	Fig. 4.32

These SP dosages and the corresponding amounts of WR resulting from these trials should be used to produce concrete for further work to get the most favourable condition of fresh and hardened concrete. The calibration graphs are believed to be a good way to obtain the right amount of SP and the corresponding WR, which produces the desired slump. Calibration graphs can produce approximate figures which could be used for further trials. These figures can be additionally adjusted to obtain the properties of the desired mix. Results revealed that standardisation of slump measurement of the superplasticized mixes is essential to eliminate any variation arising from the measurement being conducted at different times. Experience, visual inspection of the mix, and engineering common sense alongside the calibration graphs are all important to ensure selection of appropriate SP and WR values.

4.9 SUMMARY

The following conclusions can be drawn from the work presented in Chapter 4:

1. The materials intended to be used in this study were tested in compliance with the current relevant standards and technological methods. Results showed their suitability for use in producing concrete mixes.

2. Results showed that RGD grains were coarser than PFA, yet similar to PFA in some aspects, particularly the chemical composition; therefore the potential exists for its usefulness in concrete.

3. The partial replacement of cement by PFA increases the normal consistency and the setting times of the binder paste (cement and PFA).

4. All types of SP used are capable of reducing the amount of water required to make consistent binder paste; water demand was reduced to 25, 23 and 21% when SP types B, A and C were used respectively. In contrast, water content was increased as PFA was substituted for cement; this can be attributed to the increase in surface area of the binder as PFA is finer than cement.

5. SP type A should be selected for the longest possible setting times when the percentage of PFA is less than 25%, and type C for larger PFA levels, type B produces the shortest possible times. The water demand of the paste and the loss of workability of concrete results support the selection of SP type A.

6. The water demand of the binder paste increased as the level of PFA was increased, depending on the percentage of PFA. For 100% PFA replacement, water demand was increased to 23%.

7. The compatibility of an SP with the concrete constituents should be tested to avoid any adverse reactions. To ensure compatibility and reduce the negative implications of higher slump loss, a simple experiment of slump measurement of concrete having the same ingredients at time intervals using the available superplasticizers, can show the best alternative; *e.g.* the most compatible superplasticizer to the available cement (and other ingredients).

8. The behaviour of concrete made with PFA is not similar to the performance of the binder paste mixes; concrete contains a large aggregate content (up to 80% by volume) that significantly influences its behaviour; therefore reliance on paste data alone could lead to misleading conclusions.

9. Slump of the superplasticized mixes was time dependent, and therefore must be standardized with respect to time; the adding time and the time of slump measurement should be kept constant when different SP contents are compared.

10. Relationships between SP and WR against slump produced calibration charts which can be used to determine approximate values of SP and WR required for producing the desired slump. Other factors such as cohesiveness, uniformity of the mix, ease of handling and compaction, segregation and bleeding properties are deemed equally important.

CHAPTER 5
RECYCLED AGGREGATE CONCRETE MADE WITH SUPERPLASTICIZER

5.1 BACKGROUND

In view of the current scale of construction and industrial wastes ending up in landfill, as introduced in Chapter 2, it was concluded that unless appropriate measures are taken to mitigate the adverse implications of increasing waste volume, the consequences could impinge upon many aspects of life. Natural environment and ecosystems will be harshly disturbed and the excessive consumption of natural reserves worldwide will increase. Therefore, there is a need to maximize the recycling of these materials. The key factor affecting widespread spreading use of RA in construction is the development of techniques that lead to improve the quality and performance characteristics of the recycled aggregate concrete. There are perhaps many available alternatives; the use of mineral and chemical admixtures such as PFA and SP seem to be encouraging options.

Throughout this study, all RAC mixes will be produced with 100% RA; this was considered to be the worst possible case. Earlier research showed that blended aggregates (NA and RA at different levels) have produced better concrete than RAC concretes. However blended concrete will be tried, on one mix; the results are presented at the end of this chapter.

Literature showed that high strength concrete is fundamentally produced by good quality natural aggregates. The strengths, commonly used in practice, are within the range of 50-90 N/mm^2, even though higher strengths are of course achievable. However, the strength required in most of the cases is normally below 60 N/mm^2 (Neville 2003). Commercial high-strength concrete mixtures are often designed to obtain 50-80 N/mm^2 at 28 days (Mehta 2002).

The vast majority of ordinary NAC concrete structures, however, are regularly designed to attain a compressive strength within the range of 25-50 N/mm^2 after 28 days. High quality NA is needlessly specified for several ordinary applications where higher strength is not needed or not a major necessity, at the very least in many elements of many structures.

This experimental study is an attempt to produce good quality recycled aggregate concretes utilising the advantages offered by modern SPs and mineral additives such as PFA and red granite dust (RGD). It is supposed that these materials can result in an RAC concrete that would be capable of developing strength and durability usually achieved by similar NAC concrete. This assumption was made on the basis of previous experience which showed that SPs and mineral admixtures such as PFA, silica fume, ground granulated blastfurnace slag, limestone powder, and many other natural pozzolans and man-made materials were successfully used to enhance the properties of NAC concrete. In this chapter, RAC concrete will be produced only with SPs.

5.2 SOME CONCRETE ASPECTS: AN OVERVIEW

Concrete will be tested to investigate its properties, and therefore some testing aspects will be reviewed briefly in the following paragraphs.

(a) Modes of failure of concrete elements

In this experimental study, concrete specimens will be loaded until failure to determine the compressive and tensile strengths. Thus it was considered essential to review the possible modes of failure of a concrete element. Past experience has shown that when a sample is loaded gradually in the laboratory to simulate the case of a real structural member, failure occurs when the imposed stress exceeded the strength capacity of concrete. Failure of concrete derives from its weakest link. Within that link, failure is usually initiated when micro-cracks are imitated, and then as a result of a load increment several micro-cracks are created and united to form larger macro-cracks which propagate to cross nearby grains and connect the other weak links of the concrete system, eventually resulting in concrete deformation and crushing.

It is well known that strong aggregate particles (the case with various natural aggregates) are capable of hindering crack propagation as they embody sufficient resistance to tension and shear. Previous research show that RAC concretes are often encapsulated by weak, friable particles and soft impurities such as timber, glass, paper, rubber, mortar, *etc.*, which tend to weaken RAC, particularly when present in large amounts. Therefore, RAC is likely to be less competent in deterring the spread of cracks and deformation. However, weaker aggregates are liable to produce more ductile concrete than stronger aggregates.

Drawn from the literatures, four distinct failure modes or deformation states are possible:

- Matrix failure

  This failure mode occurs when the compressive strength of the matrix is exceeded while the strength of the aggregate particles is not fully used.

- Interlocking failure

  This type of failure occurs when the bonding tensile strength in the transition zone between the matrix and the aggregate grains is exceeded. In such cases, the aggregate usually have, in comparison with matrix, relatively high strength and their full potential strength did not used.

- Aggregate failure

  Aggregate failure happens whenever the tensile stress across the aggregate particle goes beyond its tensile strength.

- Combined interlocking and aggregate failure

In rich concrete mixes, in terms of cement content, the possibility of failure within the paste matrix alone, is very rare, as this link is usually very strong, and therefore, the failure plane has to pass through the matrix-aggregate interface or through the aggregate (Beshr et al. 2003). Full strength potential of the coarse aggregate would be developed when aggregate grains are sheared, under loading.

(b) Compressive strength

At 28 days, a concrete made with ordinary Portland cement and kept in normal curing conditions will develop about 75% of its strength (Jackson 2005). In this work, specimens were made and tested for compressive strength in accordance with BS EN 12390-3: 2002. The strength characteristics of each cube were tested in compression at a loading rate providing a stress increase between 0.2 N/mm^2 *per* second and 1.0 N/mm^2 *per* second on a 300 kN Avery Denison Universal Testing Machine. Three cubes for every curing age will be crushed, and the average compressive strengths will be determined.

(c) Splitting tensile strength

It is well known the tensile strength of concrete can be experimentally determined by uniaxial tensile test, split cylinder test, split cube test and beam test in flexure. A cube splitting tensile test involves the loading of concrete specimens as show in Fig. 5.1 in accordance with BS 1881-117: 1983 and BS EN 12390-3: 2000. An increasing load is applied to the specimen continuously, without shock, at a rate of 0.02 - 0.04 N mm^{-2} s^{-1}. This rate is maintained until failure and the failure load is recorded. From the failure load, the tensile splitting strength f_{ct} can be calculated using the formula:

$$f_{ct} = \frac{2F}{\pi LD} \quad (N/mm^2) \qquad \text{(Eqn 5.1)}$$

Where, F is the failure load (N), L is the length of specimen (mm), and D is the depth of specimen (mm).

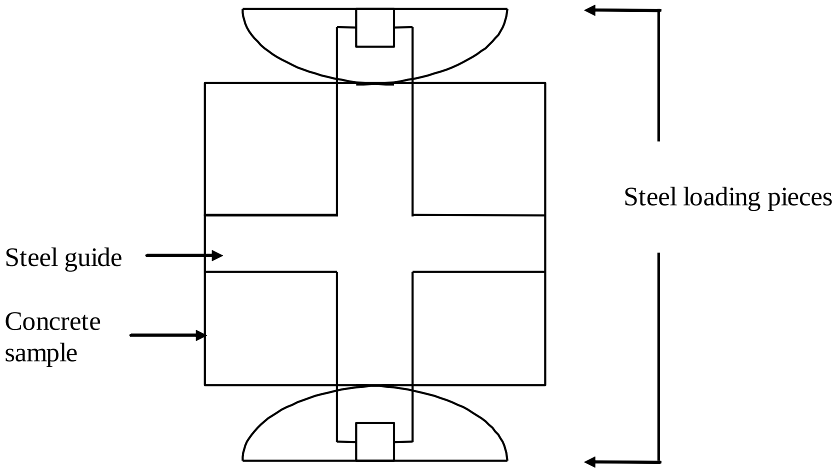


Fig. 5.1 Loading arrangement for tensile splitting strength

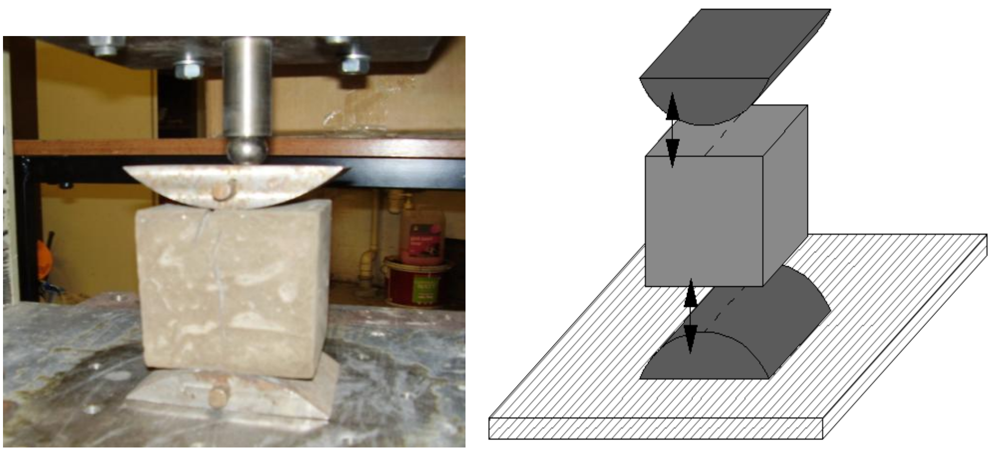


Fig. 5.2 Loading of a cube for tensile splitting testing (L) and cube alignment (R)

Concrete is not normally designed to resist direct tension (Neville 1995), but the knowledge of tensile strength is of value if estimating the load under which cracking will develop. Therefore tensile strength needs to be measured for the assessment of concrete quality. Three cubes at each curing age will be tested, and the average tensile strengths will be determined.

(d) Flexural strength

The tensile strength is measured by subjecting a plain rectangular concrete beam to flexure. Flexural strength of hardened concrete is determined by testing a concrete beam using a two-point loading method in accordance with BS 1881-118: 1983 and BS EN 12390-5: 2000. The beam is simply supported on a pair of steel rods of 25 mm diameter located at a distance of 50 mm from the edges and welded on a rigid steel channel-shaped plate. Two additional, similar, rods were placed at the third points between the supports on top of the beam. The loading arrangement to determine the flexural strength of concrete is shown in Fig. 5.3. The load was applied without shock at a rate of 0.06 ± 0.04 N/mm^2 *per* s until failure occurred. The flexural strength f_r (modulus of rupture) is related to the load and cross-sectional area and is calculated using Eqn 5.2:

$$f_r = \frac{FL}{d_1 d_2^{\,2}} \qquad \text{(Eqn 5.2)}$$

Where F is the failure load, L is the length of the beam (distance between supporting rollers) and d_1 and d_2 are the width and depth of the specimen respectively.

In this study, measuring the flexural strength of concrete; beams of size 400 × 100 × 100 mm will be produced using recycled aggregate: a set of concrete beams containing natural aggregate will also be produced for comparative purposes. The beams will then be cured in a laboratory water tank until tested to determine their flexural strength.

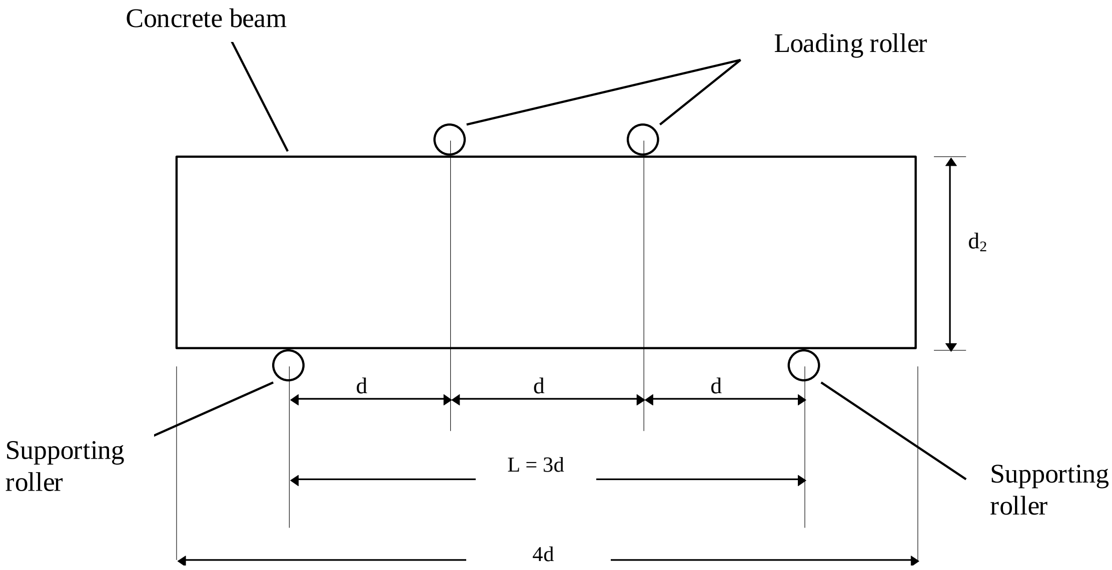


Fig. 5.3 Arrangement of loading test piece (two point loading)

Each test beam's size is 400 mm in length with a cross-sectional area of 100 × 100 mm, giving d = 100 mm in Fig. 5.3. Eqn 5.2 is appropriate only if fracture occurs within the central third of the beam.

(e) Modulus of elasticity

From the point of view of the design and behaviour of structures, not only compressive strength but modulus of elasticity is important (Zhou 1995). The static modulus of elasticity (E_s) of concrete is obtained by dividing stress by strain where stress is a result of applying external load on the surface area of the concrete sample and strain is the deformation caused by external load. Therefore, E_s is an indication of a concrete's ability to retain its original shape after being subjected to stress. As E_s is a ratio of stress over strain, the strain is the denominator and thus mathematically E_s would be increased when the deformation is low (decreased strain), as a result a higher modulus concrete is more brittle than concrete with low E_s. This value is important in designing structural elements; it is also employed in estimating the deflection of elements when subjected to loading. The static modulus of elasticity of a material under tension or compression is obtained by the slope of the stress-strain curve under uniaxial loading.

Concrete with a higher strength will most likely suffer less deformation (E_s and f_{cu} are correlated), and therefore have higher E_s than a concrete with a lower strength which should undergo more deformation. This means concretes with high strengths should be more rigid or brittle (and less forgiving with respect to structural failure suddenness) than concretes with low strength.

Since the stress-strain curve of concrete is non-linear (Neville 1995), the secant modulus was obtained for mixes under study. The secant modulus is the slope of the line drawn from the origin to the point on the curve corresponding to 40% of the failure stress of the cube *i.e.* $0.4f_{cu}$; this value is also known as the yield stress; stress-strain relationship exhibits linear behaviour below this stress level. The E_s values have been determined on a stack of three cube specimens under uniaxial compression.

The deformation (strain) was measured by two mutually perpendicular strain gauges fixed on the middle cube to measure the horizontal and vertical strains. In this study, the modulus of elasticity of the concrete mixes was measured to investigate the effect of replacing NA with RA and the use of PFA or RGD, and SP on this property. For the determination of elastic modulus, the concrete cube was first loaded to one-third of the average compressive strength of three cubes, for each individual mix, for two cycles and then load was applied continuously without shock at a rate of 180 kN/ minute until failure was reached. Fig. 5.4 shows a sample of a stack loaded for the determination of elasticity modulus.

Note:
A strain gauge used in the experiment was manufactured by Dearborn, USA; wired as quarter-bridge circuit into System 5000 data-logger; characteristics are:

- Type: Wire 26ga -19 × 38 – 600V- 0.01PVC - 105C No 172619.
- Resistance of bridge circuit = 120 Ω.
- Gauge factor = 2.10.
- Dummy gauge not needed as it is a thermal compensating gauge.

Fig. 5.4 Concrete cubes prepared for modulus of elasticity test

This chapter will present the results of the experimental research work carried out, as a part of this wider study involving recycled coarse aggregate, to assess the potential improvements of superplasticized RAC.

(f) RAC: the use of SPs

It is well known that due to their higher porosity, as compared to natural aggregates, RA absorbs more water than NA, in particular the recycled aggregates derived from cement and clay crushed bricks. It is sometimes recommended to pre-soak these types of aggregates, before mixing to prevent the concrete from becoming too dry which affects the workability of fresh concrete (Hansen & Narud 1992; Khaldon 2005; Khalaf & DeVenny 2004). Water absorption for brick aggregates lay within the range of 22 - 25% and that for crushed concrete is 5-10% (Wrap 2007); this amount was reportedly even lower when the level of attached mortar reduced (Hansen & Narud 1992). However, it is practically difficult to ensure saturated surface dry condition (SSD) for all batches of aggregates. Instead, with the help of superplasticizing admixtures; *i.e.* SPs, acceptable levels of workability can be achieved and the possible impact of RA on absorption can be covered or eliminated. Only small quantities of SP are required, typically 1 to 2% by weight of cement or binder; the overall effect is one of greater lubrication and increased fluidity (Illston & Domone 2001). The RA which will be used, throughout this study, is created from crushed concrete (not from bricks) for which the average absorption capacity was determined to be about 4.6% (see Chapter 4 for more details). In view of the preceding information the RA will be used as supplied from the recycling plant, *i.e.* will not pre-soaked before mixing.

5.3 RAC PRODUCED WITH SUPERPLASTICIZERS

In this part of the study; a total of nine mixes were produced; three standard mixes designed to have strengths of 40, 50 and 60 N/mm^2, and another six mixes (three for NAC and three for RAC concrete) were produced with SP type A at a level of 0.8% by mass of the cement content. The aim was to investigate the influence of the new polymer-based superplasticizer on the strength of RAC concrete.

Table 5.1 displays details of the mix proportions, while the complete mix design sheets of the three standard mixes, designed in compliance with Building Research Establishment mix design method (BRE 1992), are shown in Appendix 2. Mixes are coded as XY-Z where X shows the type of aggregate; Y the characteristic strength after 28 days and Z the type of SP. For instance, RAC60-A is a recycled aggregate concrete mix designed to attain 60 N/mm^2 after 28 days in which SP type A was used. Concrete mixes grade 40 and 60 were designed to have a slump of 30-60 mm and concrete grade 50 was designed to have lower slump of 10-30 mm.

It should be pointed out that the mixes produced at this stage were intended to explore the possible range of strengths that may be achieved when RA is used to produce RAC concrete. Based on results, the testing programme will then be extended to cover other concrete characteristics.

The control mixes were produced with the concrete component obtained from the mix design. However, it was observed that control mixes tended to produce slumps near to the minimum limit of their design slump, but were considered satisfactory.

For mixes other than the control mixes, SP type A was used at a constant rate of 0.8 % of cement content. The mixing water was reduced to maintain the design slump (using the calibration charts of Ch. 4), as the SP significantly increases the fluidity and flowability of concrete. In all cases, the first trial mix was mixed with the higher water reduction value of 40% of the standard water; then the water content of the mix was gradually increased until an acceptable slump was obtained, however, the first trials produced dry concrete. For each mix, trials (the gradual increase of water and measurement of the corresponding slump) time was kept below 15 minutes; when exceeded, for one reason or another, similar new mix was produced, and trials continued. On the basis of trial and error, it was concluded that the water content needed to be reduced to about 30, 16 and 25% for mixes NAC40, NAC50 and NAC60 respectively. Results indicate that for high slump mixes (30-60 mm), the SP works very well enabling higher water reduction as compared to low slump mix NAC50 (10-30 mm).

Concrete was mixed in a suitable horizontal tank laboratory mixer, cube samples were cast in moulds 100 × 100 × 100 mm, compacted on a vibrating table, stripped after 24 hours and water tank cured at 21 ± 2°C. The specimens were tested in their wet condition soon after their removed from the tank. Three samples were tested and an average result for all cases was obtained. Mix proportions of standard mixes are shown in Table 5.1.

Table 5.1 Mixes proportions of concrete grades 40, 50 and 60

Mix	Mass (kg/m^3)				
	Cement	Water	Fine aggregate	Coarse aggregate	Wet density
NAC40 (control)	457	210	660	1073	2400
RAC40	457	210	590	1093	2350
NAC50 (control)	425	170	550	1285	2430
RAC50	425	170	530	1240	2365
NAC60 (control)	636	210	480	1074	2400
RAC60	636	210	465	1039	2350

The successful trial mixes are given in Table 5.2.

Table 5.2 Standard mixes and the successful trial mixes with SP

Mix	Water (kg/m^3)	SP (type)	SP (%)	SP (kg/m^3)	Water Reduction (%)	w/c	Slump (mm)	
							Design	measured
NAC40 (control mix)	210	-	-	0	0	0.46	30-60	35
NAC40-A	147	A	0.8	3.6	30	0.32	30-60	60
RAC40-A	147	A	0.8	3.6	30	0.32	30-60	45
NAC50 (control mix)	170	-	-	0	0	0.40	10-30	15
NAC50-A	142.8	A	0.8	3.4	16	0.34	10-30	45
RAC50-A	142.8	A	0.8	3.4	16	0.34	10-30	30
NAC60 (control mix)	210	-	-	0	0	0.33	30-60	40
NAC60-A	157.5	A	0.8	5.1	25	0.25	30-60	50
RAC60-A	157.5	A	0.8	5.1	25	0.25	30-60	35

Water reduction is the amount of water that produces a slump within the design limit.
Average slump was measured to the nearest 5 mm.

Concrete cubes were cast from these trials and crushed to determine the strengths. Results are summarised in the paragraphs to follow.

(a) Compressive strength

Results of compressive strengths at different ages are listed in Table 5.3. Results showed that the strengths achieved by concrete mixes were relatively high, particularly for the NAC concrete specimens. The influence of SP is very obvious; the mixing water was reduced when SP was introduced to maintain the consistency, and therefore the w/c ratio was decreased resulting in higher strengths. When compared to control mix which was produced with a relatively high w/c ratio (0.46), the strengths were significantly improved. In addition, the excellent quality of NA, used with all NAC concrete mixes, and the high cement content also has a role in producing higher strengths. For instance in the mix NAC60-SP, the cement content was 636 kg/m^3 and the w/c ratio was 0.25 resulting in a maximum strength of 110 N/mm^2 (the average of three samples was 100 N/mm^2).

Table 5.3 Compressive strength of concrete mixes with SP

Mix	Target mean strength (N/mm^2)	Compressive strength of concrete (N/mm^2) after a period of : (days)				CV 28 days (%)
		7	14	28	56	
NAC40 (Control mix)	53	41.7	65.6	70.6	75.1	2.8
NAC40-A	53	76.3	84.6	95.4	95.0	2.0
RAC40-A	53	59.5	60.6	64.5	66.7	4.0
NAC50 (Control mix)	63	52.4	67.5	69.3	71.2	1.96
NAC50-A	63	63.8	68.3	74	89.7	5.75
RAC50-A	63	59.6	61.8	64	72	7.51
NAC60 (Control mix)	73	60.4	81.9	84.3	88.4	3.6
NAC60-A	73	93.7	95.0	100	108.5	1.5
RAC60-A	73	65.3	66.0	67.5	70.4	6.0

The compressive strength graph for concrete mix grade 40 is displayed in Fig. 5.5. Comparison between NAC and RAC concrete of this mix, with respect to development of strength over age, and the achieved compressive strengths can be readily made.

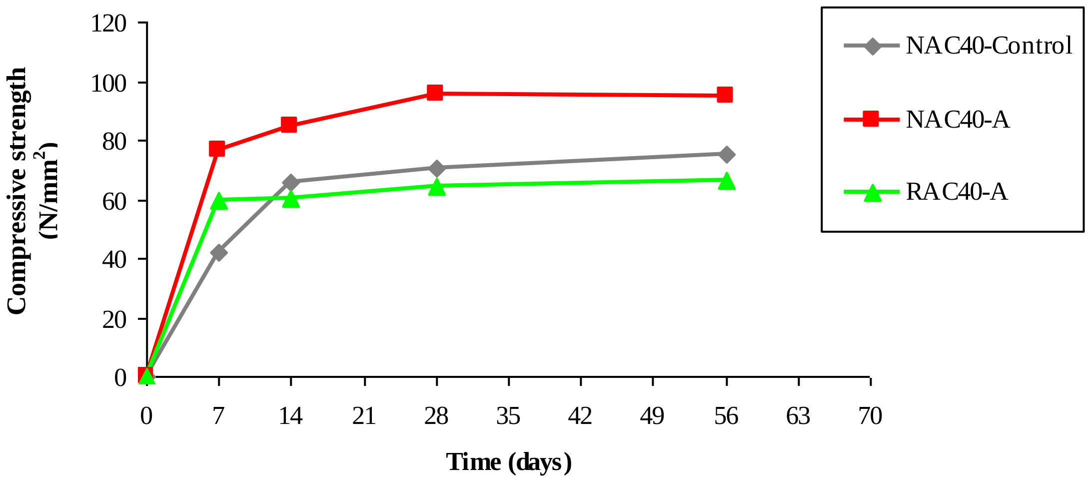


Fig. 5.5 Development of compressive strength of RAC as compared to NAC concrete

Fig. 5.5 shows that RAC and NAC have followed the same trend in strength development; strengths were gradually increasing over time; however, insignificant increase was observed after 14 days in all cases.

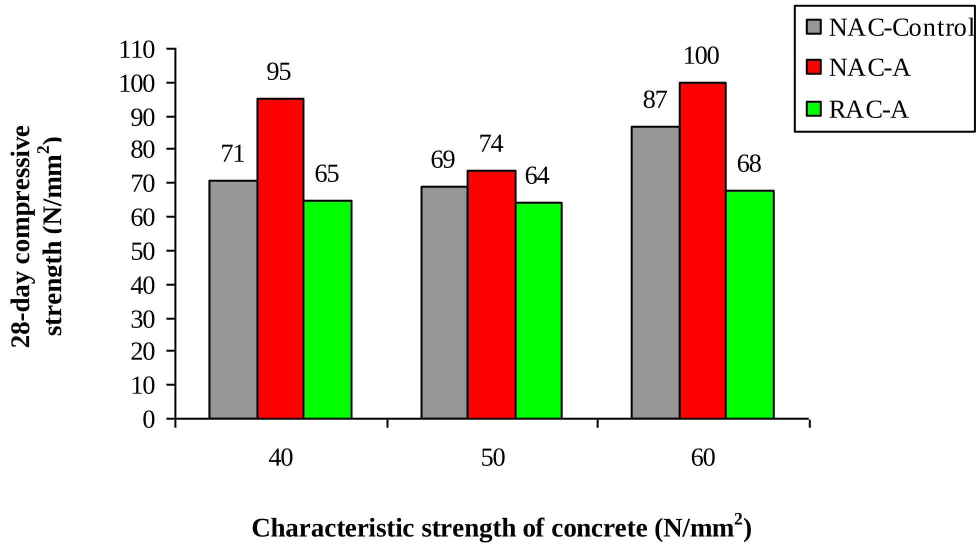


Fig. 5.6 Compressive strength for NAC and RAC mixes produced with SP type A

The concretes produced were designed to achieve different strengths and different slumps, and therefore, the comparisons are possible only between the mixes with natural and recycled aggregates within the same concrete grade. Fig. 5.6 shows that all RAC and NAC concrete mixes achieved their characteristic and target mean strength after 56 days (53, 63 and 73 N/mm^2 respectively), while only the 28 day compressive strength of recycled aggregate mix RAC60-A was slightly below the target mean strength by around 7.5%.

Despite the higher amount of cement in concrete grade 60 relative to the other two mixes *i.e.* 636, 425 and 457 kg/m^3 for concrete grades 60, 50 and 40 respectively, and the decrease in w/c ratio; RAC concrete mixes attained almost similar ultimate strength; 67.5, 64.0 and 64.5 N/mm^2 for grades 60, 50 and 40 respectively. The compressive strength of RAC concrete achieved in these experiments has probably reached the limiting strength of the RA used, and the concrete strength did not benefit very much from further improvement of the matrix strength. Although RAC concrete mixes have achieved good strengths when compared with usual strengths of ordinary concrete; in all cases and ages, the compressive strength of RAC concrete was, as expected, lower than the similar NAC mixture. This situation is well known to happen even with NAC concrete mixes. Higher strengths of concrete can be drastically reduced by weaker aggregates (Zhou 1995).

(b) Splitting tensile strength

Splitting tensile strengths were measured after 28 days and the results are displayed in Table 5.4.

Table 5.4 Splitting tensile strength of concrete mixes with SP (without PFA)

Mix	Splitting tensile strength (N/mm^2) after 28 day	CV at 28 days (%)
NAC40 (Control mix)	3.64	1.2
NAC40-A	5.03	1.1
RAC40-A	3.82	16.2
NAC50 (Control mix)	4.70	1.41
NAC50-A	5.78	16.6
RAC50-A	4.79	0.5
NAC60 (Control mix)	3.31	3.7
NAC60-A	6.43	4.8
RAC60-A	2.55	14.0

Results are an average of three samples.

Splitting tensile strengths of NAC and RAC mixes are displayed in Fig. 5.7. Results show that the tensile strengths of RAC were comparable to the reference mix result, but lower than that of similar NAC for all concrete grades at all ages. This result however is not in agreement with some previous research work as the quality of NA (crushed granite) used was much better than that of the RA. Again similar to compressive strength, tensile strength did not increase as a result of superior strength of the concrete matrix.

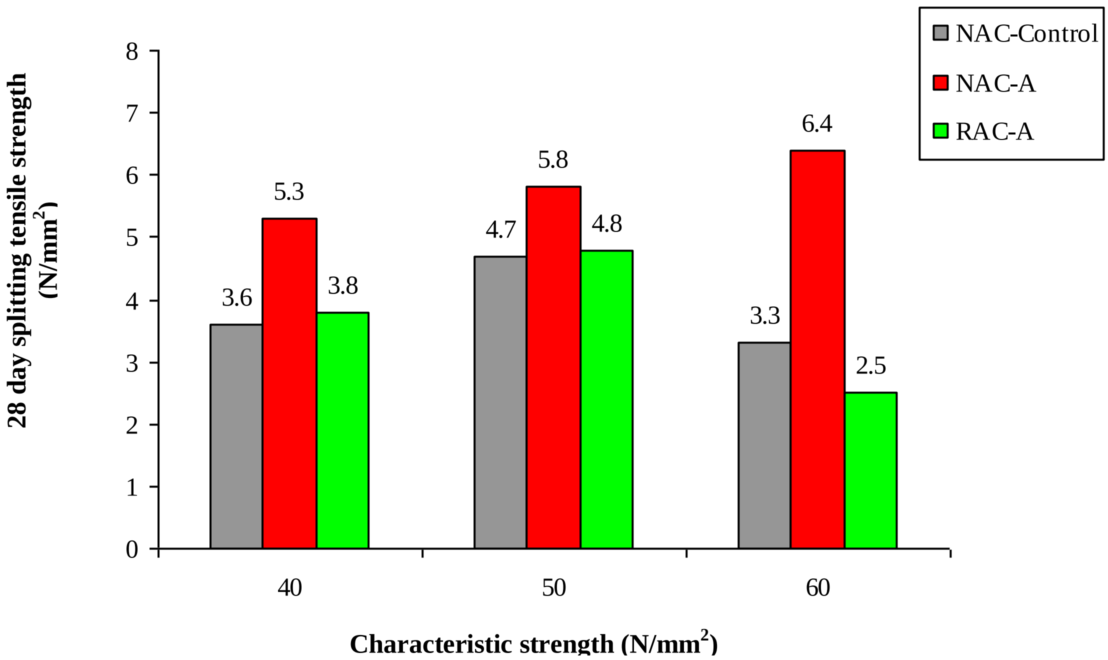


Fig. 5.7 Splitting tensile strength for NAC and RAC mixes of different grades

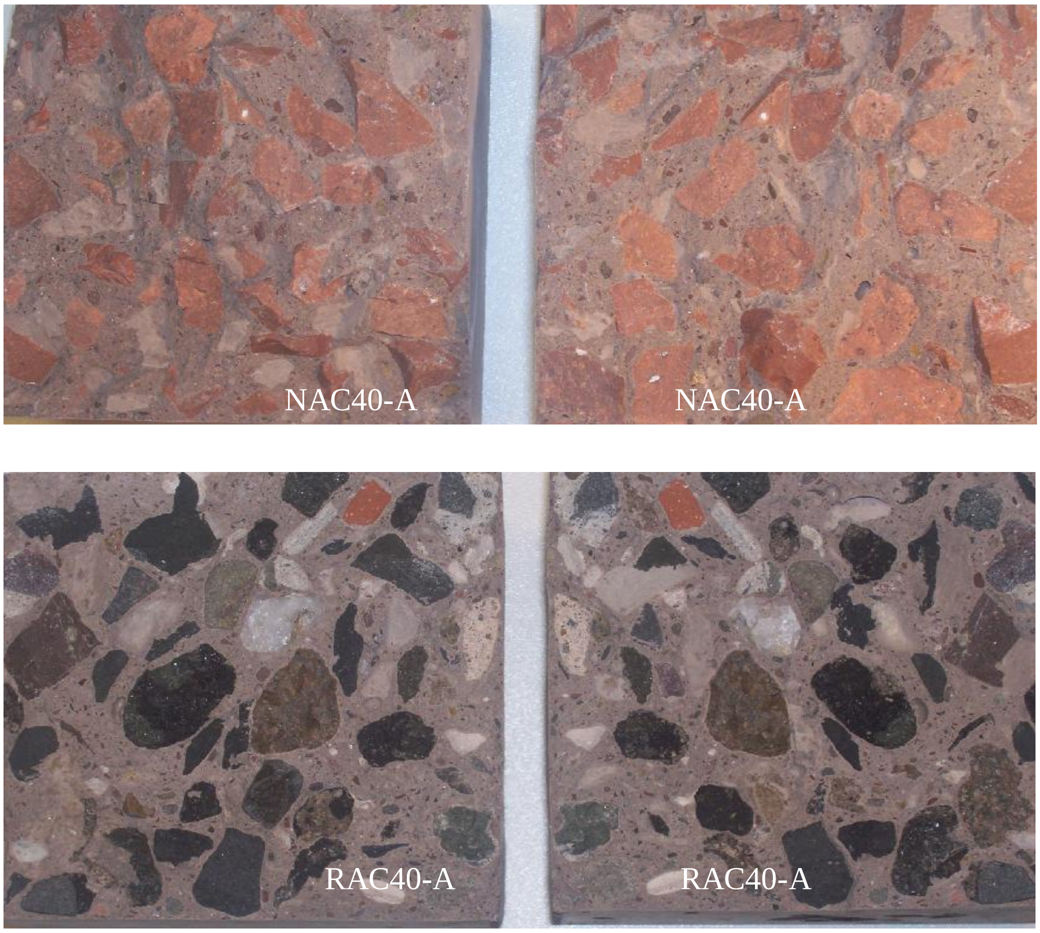


Fig. 5.8 Fracture surface of samples NAC 40-A and RAC 40-A

Close inspection of fractured surfaces of the tested samples shows that the failure was through the aggregate particles, producing two similar faces with no or little bond failure observed indicating the contribution of aggregates on the ultimate strength. This is simply indicating that the aggregate was the weakest link in the concrete system. Samples are shown in Figs 5.8 to 5.10.

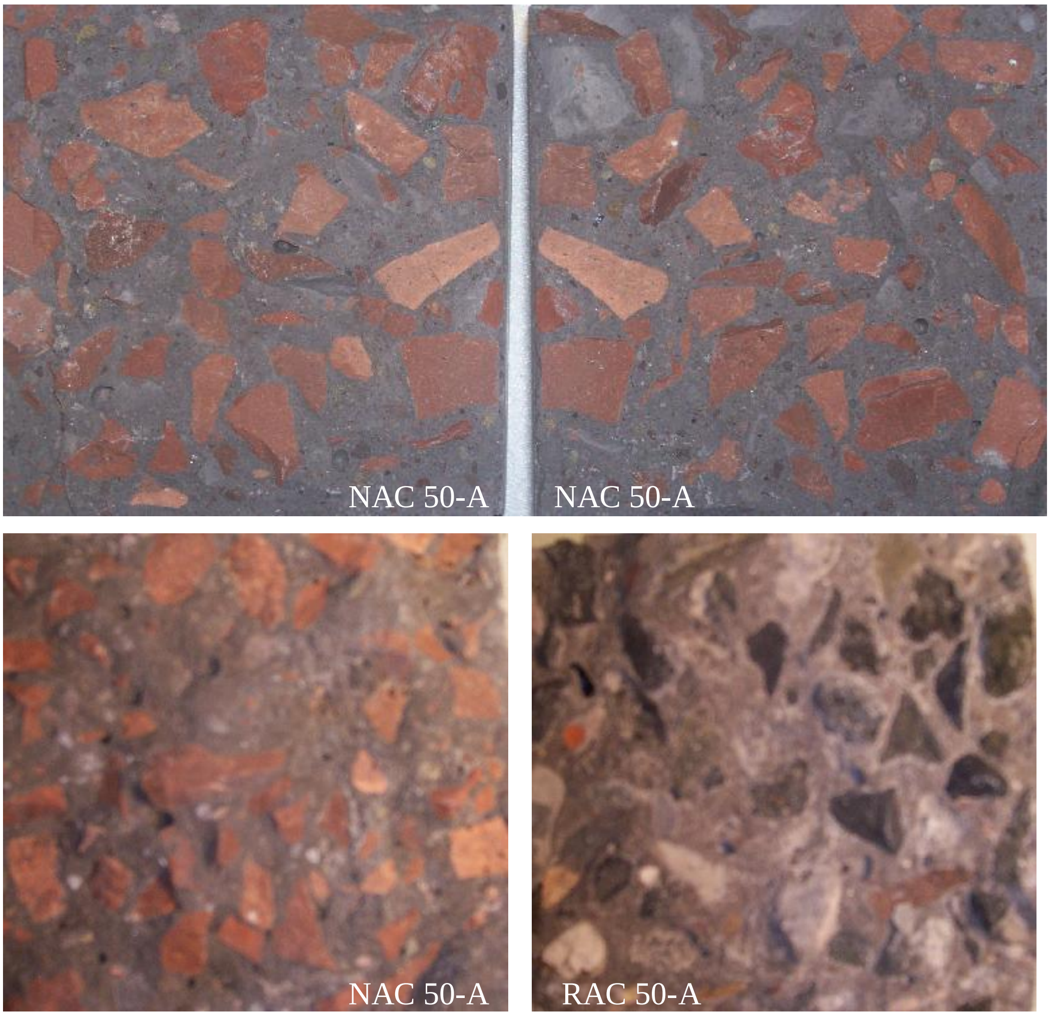


Fig. 5.9 Fracture surface of samples NAC 50-A and RAC 50-A

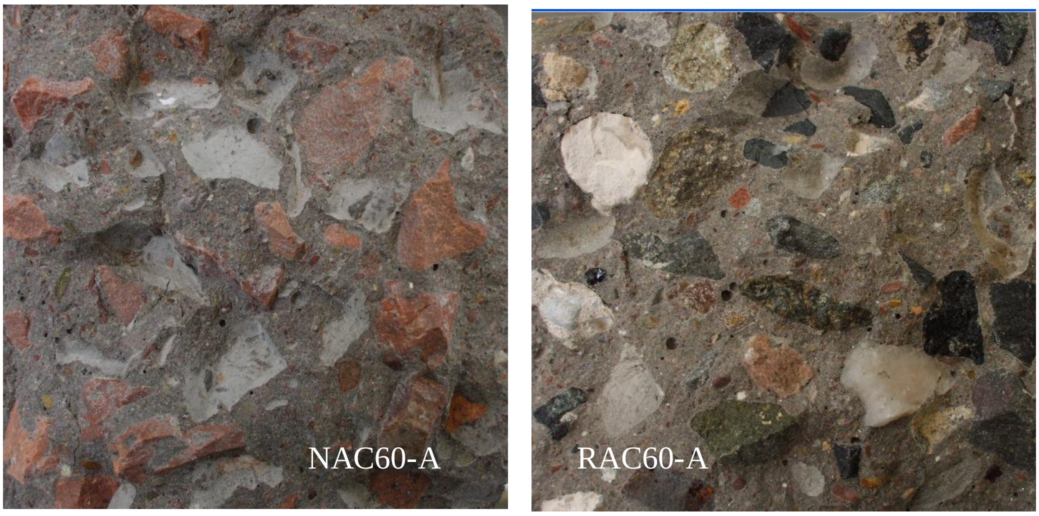


Fig. 5.10 Fracture surface of samples NAC 60-A and RAC 60-A

(c) Density of hardened concrete

The density of hardened concrete cubes was measured just before crushing, by weight and relative density apparatus as shown in Chapter 4 in accordance with BS 1881: Part 114: 1983. The range of concrete density for normal weight concrete is between 2200-2600 kg/m^3; typically it is about 2400 kg/m^3 (Neville 1995). The measured concrete densities were more or less those average values frequently obtained. Results are displayed in Table 5.5.

Results revealed that the densities of the RAC were close to NAC densities and both lay within the aforesaid range. All concrete densities are comparable and the concretes can be considered as normal weight materials.

Table 5.5 Bulk density of hardened concrete mixes with SP (without PFA)

Mix	Average bulk density of concrete samples (kg/m^3) measured after a period of (days):			CV -28 day (%)
	7	14	28	
NAC40 (Control mix)	2449	2461	2477	1.3
NAC40-A	2391	2390	2444	0.35
RAC40-A	2440	2446	2456	0.85
NAC50 (Control mix)	2382	2385	2400	0.5
NAC50-A	2340	2320	2312	0.27
RAC50-A	2345	2350	2348	0.11
NAC60 (Control mix)	2445	2482	2501	1.4
NAC60-A	2428	2454	2446	0.19
RAC60-A	2449	2447	2472	0.8

Density results are an average of three samples.

Literature and past experience showed that the density of hardened NAC concrete can be varied considerably, depending upon the parent rock from which the concrete aggregate was derived. RAC densities can also be wide-ranging, due to attached old mortar on the surface of RA particles and the presence of impurities. However, in general RAC density was reportedly less than that of the similar NAC concrete, particularly when RAC is produced with an aggregate created from crushed cement or clay bricks, as a result of their relatively higher porosity. It is also the case when lightweight aggregates, produced from crushed concrete, are used for RAC concrete.

In this investigation, RAC concrete has shown comparable results to NAC as the three RAC concrete mixes were able to achieve strengths above their design characteristic strengths at 28 days. The produced concrete can therefore be ranked as good quality RAC given that the achieved compressive strengths were 64.5, 64 and 67.5 N/mm^2 for concrete grades 40, 50 and 60 respectively. Higher strengths were attained at the age of 56 days, and are predicted to increase with concrete age as a result of continuous cement hydration. The coefficients of variation (CV) for both compressive and tensile strengths of RAC concrete were not large. However, RAC coefficients were comparatively higher than similar NAC concrete reflecting the variability of RA, even though RA was supplied from the same source. However, the RA used in this study was produced in a recycling facility under good quality control therefore the designed RAC concrete was good too. Low quality RA in terms of strength, absorption capacity, impurities content including adhered and loose mortar, chlorides, sulphates and bitumen, dust, soft bricks are not expected to produce good quality RAC; quality though could be improved by eliminating or minimizing some or all (if possible) of the degrading factors, and by applying very strict quality assurance and control policy. For low strength applications, low quality RA could be used when satisfying other requirements and considered to fit for the purpose at hand.

As relatively small number of samples was tested in this part of the study, results can not be statistically interpreted to draw firm conclusions. But, these mixes were basically produced, as a preliminary check, to explore or examine the hypothesis made *i.e.* quality of RAC concrete will be improved by incorporating admixtures. Results will be used as a base for further testing incorporating RA and other admixtures. However, broadly speaking, it is evidently possible, with the help of SP, to produce good quality RAC concrete, which can be utilized for construction including structural concrete applications.

5.4 MIXES MADE WITH BLENDED AGGREGATES

Literature showed that blended aggregate concrete could lead to improved strength RAC concrete. In an unpublished project (McNulty 2005) carried out at concrete unit at Edinburgh Napier University, in which aggregates similar were used, it was revealed that the best mixing option for blended recycled aggregate was 50% substitution. In this project, only the substitution level was studied and no SPs were used.

In this investigation, 50% RA trial mixes were produced with and without SP type A. The SP dosage and the corresponding water reduction were chosen based on the result of Chapter 4 Section 4.8 (e) and Table 4.27; that is 0.8% SP and 16% WR. The control mix was designed to achieve a slump of 10-30 mm and strength of 50 N/mm^2 after 28 days. The results are given in Table 5.6.

Table 5.6 Strengths of blended RAC

Mix	Slump (mm)	Compressive strength after: (N/mm^2)			Splitting tensile strength after: (N/mm^2)			Flexural strength after: (N/mm^2)	
		28 d	56 d	90 d	28 d	56 d	90 d	28 d	56 d
NAC-CM	15	71.3	73.2	76	4.11	3.92	4.51	7.32	10.40
RAC-CM	15	53.4	57.6	60.4	3.71	4.04	4.16	7.93	7.81
NA50-RA50	10	65.6	71.2	67.2	4.02	4.42	4.64	7.33	6.61
NA50-RA50-16WR-0.8SP	25	73	75.41	77.3	4.45	4.84	5.5	8.34	8.03

Results are an average of three samples.

The results clearly indicate that the strengths of blended RAC were improved, when compared to standard mixes, as a result of low w/c ratio and the use of SP. The mix with SP had shown better workability as compared with no SP mixes. Fracture surface of a sample is shown in Fig. 5.11; little bond failure was observed indicating considerable contribution of aggregate grains to the strength of concrete.

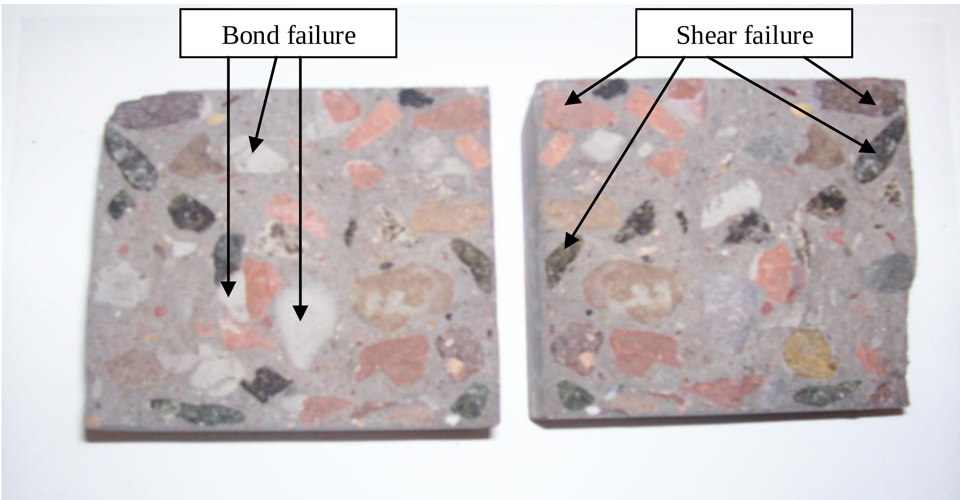


Fig. 5.11 Fracture surface of NA50-RA50 (blended aggregate concrete)

5.5 SUMMARY

From the work carried out in this chapter, the following conclusions were drawn:

1. Although a 100% RA was used for coarse aggregates, it proved possible to produce RAC concretes capable of achieving the design slumps and compressive strengths within the range of 64-67 N/mm² and 66-72 N/mm² after 28 days and 56 days respectively. The addition of SP allowed the water content to be reduced and helps to produce workable concrete. Comparable tensile

strengths were also obtained, and strengths improved with age. Higher strengths could have been produced if high-strength cement was used (the compressive strength of cement used was 42.5 N/mm^2 after 28 days).

2. Results showed that concrete strengths were influenced by aggregate type. Although comparable, the cube strength of all RAC concerts were below similar NAC at all ages, this may be attributed to the superior quality of natural granite aggregate used to create NAC concrete mixes, and to the presence of deleterious materials in RA. For relatively rich RAC concrete mixes; *i.e.* having higher cement content, strength is evidently controlled by the RA, and RAC did not benefit much from the improved strength of the paste matrix; perhaps the strength of aggregate is limiting the ultimate strength.

3. For relatively rich RAC concrete mixes; *i.e.* having higher cement content, strength is evidently controlled by the RA, and RAC did not benefit much from the improved strength of the paste matrix; perhaps the strength of aggregate is limiting the ultimate strength.

4. Strengths of RAC concrete were improved when blended with good quality aggregate.

5. Strengths of RAC concrete achieved were promising and encouraging for further investigations.

CHAPTER 6
RECYCLED AGGREGATE CONCRETE MADE WITH PARTIAL REPLACEMENT OF FINE AGGREGATE BY PFA

6.1 INTRODUCTION.

PFA was used in previous research, either as a substitute for cement or less frequently fine aggregate, the focus was on concrete mixes made with NAC concrete, while RAC concrete was rarely tested or even considered. Therefore, in this study of PFA replacement's viability, for emphasis the two cases were examined. In this chapter PFA was used as a substitute for the fine aggregate at different levels to produce RAC concrete, whereas it was used as a cement substitute in Chapter 7. The author believes that RAC concrete performance characteristics may be enhanced in a similar way to NAC concrete. In addition to PFA's influence on concrete performance, the strength and durability of RAC can perhaps be improved, just like NAC concrete, by lowering the w/c ratio and using water-reducing SPs. An SP will provide sufficient workability of fresh concrete at low w/c ratio and in turn is predicted to improve the strength of both the cementitious matrix of concrete and the interfacial zone between the aggregate and the matrix.

The aim of the experiments herein was to determine the possible strength range that can be achieved using 100% recycled aggregate when fine aggregates in the standard mix were partially replaced with fly ash then compared to the same mixes but with a natural aggregate. The two aggregates were not combined *i.e.* not blended in these experiments as this would not give the maximum potential strength for a 100% recycled aggregate mix.

This is an example of waste by-product becoming a valuable commodity. Green energy sources produce no fly-ash: like fossil fuel, it will run out. This makes a strong case for recycled aggregate concrete: economic practicality and eco-sense combine well.

Previous research presented in Chapter 3 demonstrated that good quality NAC concretes were successfully produced when PFA was substituted for cement at different percentages (refer to Section 3.9 for more details). However, in this situation, it is convenient to remind the reader that practical and efficient levels of cement replacement with PFA, for different types of concrete, are well established. The most important may be the USA's coded limitation (ACI 1988), in which PFA replacement was restricted to 15-25% of cement content, and the recommendation (Malhotra 1990) to limit the amount to 30%. Previous studies (Mehta 1986 and 1988; Carette *et al.* 1993; Poon & Wong 2000; Bouzoubaâ & Fournier 2003; McCarthy & Dhir 2005) have shown that acceptable quality concrete produced with high-volume fly ash was however also possible.

Perhaps because PFA is not usually used to substitute fine aggregate in concrete mixes, the optimum replacement level has yet to be determined. The establishment of the most advantageous and safe level of PFA sand replacement may therefore be useful for aspects of design, mixing and handling of this type of concrete.

It is well known that the mix proportion, in practical use, usually depends on the specific requirements of the particular job on which the concrete is to be used. In this research the use of the end product is not specified. Therefore, several RAC mixes, as will be detailed later, were tested for fresh and hardened properties of concrete and compared against NAC control mixes. Standard mix design sheets for NAC and RAC concrete are detailed in Appendix 2. This chapter will present the results of the experimental research carried out to assess potential improvements of superplasticized RAC when PFA is used to partially substitute fine aggregate, as a part of the author's wider study of recycled coarse aggregates. However, more background materials will be introduced here and the engineering /scientific material will follow.

6.2 PROPERTIES OF RAC MADE WITH PARTIAL REPLACEMENT OF FINE AGGREGATE WITH PFA

(a) General: superplasticizers, PFA and cement

It should be noted that all the three SPs (types A, B and C, introduced in Chapter 4) will still be examined at this stage instead of using just the selected one (SP type A). The reason was to examine whether or not SP type A was also the best option from a strength point of view. This way, the setting times, slump loss over time, and the strengths are all taken as decisive factors affecting the selection of the most appropriate SP type.

It was concluded, in Chapter 3, that the reaction of PFA with lime, liberated from cement hydration, produces a cementitious material in the presence of moisture. This contributes to strength, particularly at later ages, in addition to its filling effect, and therefore, the amount of cement (the most expensive component of concrete) in mixes containing PFA, *i.e.* mixes other than the control were reduced to 25% cement content for economic reasons; therefore producing cheaper concrete. According to manufacturer recommendations, suitable dosages of superplasticizers are 0.6-1% for type A, 0.4-0.8% for type B and 2-3% for type C, but the manufacturer also strongly recommended that the optimal dosage should be chosen based on trials with the material at hand, however at this stage average values for superplasticizers were assumed, taking manufacturer recommendations into account, as 0.8%, 0.6%, and 2.5% for SP types A, B, and C respectively.

With respect to PFA fine aggregate replacement, the question arising at this point is about the optimal replacement level. In this context, from a number of research papers reviewed in Chapter 3; the general impression formed was that a wide range of replacement was possible. However, it may worthwhile to restate examples here; the level of PFA replacement ranged from 5-60% (Mangaraj & Krishnamoorthy 1994), 5-15% (Dhir *et al.* 2000), 10-50% (Rafat 2003a), and 20-60% was used by (Rajamane *et al.* 2007). Reference can be made to Section 3.9 (d) ii for more details.

(b) RAC concrete mixes

To achieve the objective of this investigation, 14 concrete mixes were prepared and tested. Two, out of the 14 mixes, were standard for NAC and RAC concrete; two mixes used SP but no PFA, while in six mixes, fine aggregate was partially replaced with PFA at four different replacement levels; in the final four mixes, fine aggregate was replaced with PFA at 50% replacement level. Therefore, all mixes other than standard mixes were superplasticized concrete; more details are given in Tables 6.1 to 6.5.

The superplasticized mixes were proportioned keeping the PFA level as the only variable to assess the influence of PFA content on fresh and hardened properties of concrete. To cover a wide range, low and relatively high PFA replacement levels were adopted *i.e.* 25, 50, and 75%. Mixes were coded as X-YZ-T where X represents the aggregate type, Y is the PFA, Z the level of PFA replacement to fine aggregate and T the type of superplasticizer (A, B, or C).

Table 6.1 Mixes in which PFA replaces fine aggregate, SP type and content

Mix	PFA (%)	SP type	SP (%)
NAC-CM (Control mix)	0	0	0
RAC-CM(Control mix)	0	0	0
NAC-PFA0-A	0	A	0.8
RAC-PFA0-A	0	A	0.8
NAC-PFA25-A	25	A	0.8
RAC-PFA25-A	25	A	0.8
NAC-PFA50-A	50	A	0.8
RAC-PFA50-A	50	A	0.8
NAC-PFA75-A	75	A	0.8
RAC-PFA75-A	75	A	0.8
NAC-PFA50-B	50	B	0.6
RAC-PFA50-B	50	B	0.6
NAC-PFA50-C	50	C	2.5
RAC-PFA50-C	50	C	2.5

PFA content is the percentage ratio of PFA to fine aggregate

Those mixes listed in Table 6.1 are:

1) Two standard mixes (without SP and PFA, one for NAC and another for RAC), which will be considered as control or reference mixes.
2) Two mixes produced with SP type A only (without PFA one for NAC and another for RAC).
3) Six mixes in which PFA is substituted for the fine aggregate of the standard mixes at percentages of 25, 50, and 75% with SP type A (Three for NAC and three for RAC).
4) Four mixes in which PFA content was kept constant at 50% replacement and SP types B and C were used so that comparison could be made with the mixes that contain 50% PFA produced with SP type A.

The standard mixes were designed to attain 40 N/mm^2 after 28 days, slump of 30-60 mm and Vebe time of 6-12 s in compliance with the Building Research Establishment's mix design method (BRE 1992). Mix proportions are shown in Table 6.2.

Table 6.2 Mix proportions of standard mixes

Material	Mass used (kg/m^3)	
	NAC	RAC
Cement	475	475
Water	190	190
Coarse aggregate	1125	1080
Fine aggregate	610	580
Wet density	2400	2325

Due to a difference in relative densities, standard mix designs for NA and RA were prepared individually. Appendix 2 shows the detailed mix design of the standard mixes which were used as reference mixes in this part of the investigation. In other mixes, PFA was substituted for fine aggregates at levels of 25, 50, and 75% while cement was reduced by 25% (see Section 6.2). A large specific surface area increases the adsorption of molecules on the surface (White 2005). The specific surface area is 10 m^2/kg and 100 m^2/kg for coarse and fine sand respectively (White 2005). The specific surface area of PFA is between 300 - 400 m^2/kg (see Table 4.16 Chapter 4); therefore it is about 40 times finer than coarse sand. In such case an amount of water could be lost due to this difference in surface areas. The relative density of a fly ash particle is 2.0 - 2.4 (Table 4.16 Chapter 4) less than that of the fine aggregate (2.6) and as a result there will be a greater volume of paste if substituted weight for weight. Concrete added materials have lower specific gravities than Portland cement, and therefore substitution of cement on a weight-for weight basis will result in a greater volume (Illston & Domone 2001). To eliminate these influences, the mixes were proportioned to take this into account. Therefore the amount of PFA was adjusted so that the quantity which produced the same volume of displaced sand was used. Accordingly, the adjusted amounts of PFA were as shown in Table 6.3 alongside the standard values.

Table 6.3 Mix proportions for mixes when PFA replaces fine aggregates at different levels

Material	Standard mixes		PFA mixes					
			25% PFA		50% PFA		75% PFA	
Aggregate type	NA	RA	NA	RA	NA	RA	NA	RA
Cement	475	475	355	355	355	355	355	355
Water	190	190	142	142	142	142	142	142
Coarse aggregate	1125	1080	1125	1080	1125	1080	1125	1080
Fine aggregate	610	580	457.5	435	305	290	152.5	145
PFA	0	0	152.5	145	305	290	457.5	435
PFA used*	0	0	132	125.5	264	251	396	376.5

Cement and water were reduced by 25% for all PFA mixes

*PFA used is the amount that produces an equal volume of the substituted fine aggregate

In all PFA mixes the only variable was the percentage of PFA in the mix, the water cement ratio was reduced by 25% to maintain the same w/c ratio as the cement content was reduced by 25%.

Due to the aforesaid difference in relative density and specific surface area of PFA and fine aggregate, the PFA amounts in Table 6.3 are adjusted, so that PFA used is the amount that produces an equal volume of the substituted fine aggregate, according to the following sample calculations:

1 For 25% PFA replacement, the standard amount *i.e.* the mix design amount of fine aggregate was 610 kg/m^3.

2 PFA level was 25% and fine aggregate was 75%, therefore the amount of PFA is 0.25 × 610 = 152.5 kg/m^3, and the amount fine aggregate is 0.75 × 610 = 457.5 kg/m^3.

3 The relative density of fine aggregate and PFA were 2.6 and 2.25 respectively, therefore, the actual amounts of fine aggregate used were 457.5 kg/m^3 and for PFA 152.5 × 2.25/2.6 = 132 kg/m^3.

Concretes were mixed in a horizontal tank laboratory mixer, the mixing time was kept at about 4 minutes, and all slumps were measured at about 5 minutes. Standard cube samples were than cast into moulds 100 × 100 × 100 mm and compacted on a vibrating table. The cube samples were de-moulded and kept under standard laboratory conditions, and tested for density, compressive strength, and tensile splitting strength after, 7, 28, and 56 days.

(i) Workability of fresh concrete

Preliminary trial mixes showed that the control mixes were slightly dry, and it was found necessary to increase the water content by 5% to achieve the design slump; the mixing water in Table 6.3 was therefore adjusted to 190 × 1.05 = 200 kg/m^3 and as a result the w/c ratio was changed to 0.42 instead of the mix design value of 0.40. PFA concrete's water was adjusted accordingly to 142 × 1.05 = 149 kg/m^3; however as SP was used for all mixes other than control mixes, water was reduced by 25% to ensure low w/c ratio; therefore the amount used was 149 × 0.75 = 112 kg/m^3 and thus the w/c was 0.32 (112/355). The mix proportions were again adjusted as in Table 6.4. These proportions were used to produce NAC and RAC concrete mixes in this part of the study.

Table 6.4 Adjusted mix proportions for mixes used to produce NAC and RAC concrete

Material	Standard mixes		PFA mixes					
			25% PFA		50% PFA		75% PFA	
Aggregate type	NA	RA	NA	RA	NA	RA	NA	RA
Cement	475	475	355	355	355	355	355	355
Water	200	200	112	112	112	112	112	112
Coarse aggregate	1125	1080	1125	1080	1125	1080	1125	1080
Fine aggregate	610	580	457.5	435	305	290	152.5	145
PFA	0	0	152.5	145	305	290	457.5	435
PFA used*	0	0	132	125.5	264	251	396	376.5
PFA and cement	475	475	487	480	620	606	750	730
w/c	0.42	0.42	0.32	0.32	0.32	0.32	0.32	0.32

It was observed in several previous studies (Liu *et al* 2000; Ke-Ru *et al.* 2001; Illston & Domone 2001; Rafat 2003a, 2003b and 2004; Neville 2003; Burak 2007; Nuno *et al.* 2007; Etxeberria *et al.* 2007) that the SP is always taken as a percentage of the cementitious material *i.e.* the sum of cement and PFA in the mix. Therefore, quantities of SPs used are as shown in Table 6.5.

Table 6.5 Amounts of SP used in the mixes

Mix	Code	PFA (%)	Cement (kg/m^3)	PFA (kg/m^3)	Sum of cement and PFA (kg/m^3)	SP type	SP (%)	SP (kg/m^3)
1	NAC-CM (Control mix)	0	475	0	475	-	0	0
2	RAC-CM (Control mix)	0	475	0	475	-	0	0
3	NAC-PFA0-A	0	355	0	355	A	0.8	2.84
4	RAC-PFA0-A	0	355	0	355	A	0.8	2.84
5	NAC-PFA25-A	25	355	132	487	A	0.8	3.89
6	RAC-PFA25-A	25	355	125.5	480	A	0.8	3.84
7	NAC-PFA50-A	50	355	264	620	A	0.8	4.96
8	RAC-PFA50-A	50	355	251	606	A	0.8	4.85
9	NAC-PFA75-A	75	355	396	750	A	0.8	6.00
10	RAC-PFA75-A	75	355	376.5	730	A	0.8	5.84
11	NAC-PFA50-B	50	355	264	620	B	0.6	3.72
12	RAC-PFA50-B	50	355	251	606	B	0.6	3.64
13	NAC-PFA50-C	50	355	264	620	C	2.5	15.5
14	RAC-PFA50-C	50	355	251	606	C	2.5	15.2

SP amount is the SP percentage times the binder content *e.g.* $0.8/100 \times 355 = 2.84$ kg/m^3

The measured slump and Vebe time for these mixes are shown in Table 6.6.

Table 6.6 Measured slump of concrete mixes

Mix	Code	w/c	SP (kg/m^3)	Measured slump (mm)	Vebe time (s)	Remarks
1	NAC-CM (Control mix)	0.42	0	45	7	Uniform cohesive mix
2	RAC-CM (Control mix)	0.42	0	40	10	Uniform cohesive mix
3	NAC-PFA0-A	0.32	2.84	40	8	Uniform cohesive mix
4	RAC-PFA0-A	0.32	2.84	35	8	Uniform cohesive mix
5	NAC-PFA25-A	0.32	3.89	45	6	Uniform cohesive mix
6	RAC-PFA25-A	0.32	3.84	45	7	Uniform cohesive mix
7	NAC-PFA50-A	0.32	4.96	35	9	Uniform, less cohesive mix
8	RAC-PFA50-A	0.32	4.85	25	11	Lack of cohesion
9	NAC-PFA75-A	0.32	6.00	20	12	Lack of cohesion
10	RAC-PFA75-A	0.32	5.84	15	14	Dry mix
11	NAC-PFA50-B	0.32	3.72	50	7	Uniform cohesive mix
12	RAC-PFA50-B	0.32	3.64	40	9	Uniform, less cohesive mix
13	NAC-PFA50-C	0.32	15.5	70	3	Shear slump, signs of segregation, lack of cohesion
14	RAC-PFA50-C	0.32	15.2	60	5	Less segregation, lack of cohesion

The mixes were workable, despite the lower w/c ratio and high volume of PFA, probably due to the influence of the ball-bearing nature of fly ash grains as well the dispersive effect of SP. The measured slumps for most of the mixes were well within or comparable to the designed slump (30 - 60 mm). The exceptions are mixes RAC-PFA50-A, NAC-PFA75-A and RAC-PFA75-A; for these mixes, the advantages of PFA particles being spherical seem to be counterbalanced by the large surface area of the grains; the increased surface area due to the increased PFA content is thought to have caused the decline of slump and the workability level, it seems that more water is needed to dampen and lubricate the grains.

The lower, and constant, w/c ratio used (0.32), which implies less water content, might have limited the full benefit from the spherical shape of the PFA particles, particularly for higher PFA content. The extra SP content due to the increased PFA content did not help very much in this situation therefore produced relatively low slump values.

The Vebe time was also comparable to the design value (6 - 12 s); when SP type C was used, slumps were marginally higher than the design value, which was attributed to the larger water content of this SP. The amounts of SP for high PFA content were relatively high, particularly when SP C was used. However, slumps achieved are suitable for many applications; concrete having slump of between 25 - 50 mm can be used for mass concrete foundations without vibration or lightly reinforced sections with little vibration (Neville 2003).

The results of the slump obtained showed that as the PFA content in the concrete mixture increases, the mixes were becoming less cohesive and the tendency for segregation to occur became greater. This however may be linked to the amount of free water available in the mix which turned out to be less (as w/c is constant) in the voids when the PFA level is increased; PFA grains are much finer than fine aggregates and therefore absorb more water. Another reason lies in the well known fact that the finer the aggregate, the less workable the concrete, therefore the increase of PFA level in fine aggregate could have an impact on the grain size distribution of aggregates which may lead to the observed situation.

It should also be noted that relatively rapid workability loss over time was observed, particularly when SP B was used; this result was in agreement with the result obtained for the setting times of the binder which shows that SP type B produced the shortest setting times. Therefore, whenever concrete is produced to those proportions it should be placed in the formwork within 45 minutes (preferably 30 minutes) to avoid adverse workability loss, as mixes will become stiff and difficult to place in addition to the possibility of segregation and honeycombing. Alternatively, depending on many factors including site conditions, another set-retarding technique may be adopted, examples are: adding SP to the mixture just before placing the concrete in formwork, employing an anti-setting agent, increasing the SP dose, *etc*. Higher initial mix design slump than the one used in this investigation for the standard mix (30-60 mm) could have yielded better results.

Another phenomenon worthy of mention is the de-moulding time; the de-moulding of concrete cubes for mixes with PFA content above 50%, regardless of the SP type, within 24 hours resulted in sample damage such as sticking of concrete to the mould, loss of edges and corners and severe damage when samples were forced out of the mould, therefore all mixes with 50 and 75% PFA were de-moulded after about 48 hours. The following paragraphs present the results, analysis, discussion, and the conclusion drawn from the data.

(ii) Compressive strength

Results of compressive strength of NAC and RAC samples are listed in Table 6.7. The results are average values of three specimens.

Table 6.7 Compressive strength of NAC and RAC concrete in which PFA replaces fine aggregates at different levels

Mix	Code	Compressive strength of concrete (N/mm^2) after:				CV* 28 day (%)
		7 days	28 days	56 days	90 days	
1	NAC- (Control mix)	52.6	68.5	72.6	74.7	2.2
2	RAC- (Control mix)	46.7	52.0	51.3	53.6	3.2
3	NAC-PFA0-A	54.7	74.5	79.1	82.2	1.2
4	RAC-PFA0-A	49.3	54.5	56.8	60.8	0.5
5	NAC-PFA25-A	61.3	72.1	80.3	85.8	4.4
6	RAC-PFA25-A	45.1	56.1	61.8	65.7	3.8
7	NAC-PFA50-A	46.8	62.5	72.6	78.2	2.0
8	RAC-PFA50-A	36.8	43.9	50.8	55.2	2.3
9	NAC-PFA75-A	33.3	50.7	62.0	64.5	4.5
10	RAC-PFA75-A	25.3	35.8	42.7	46.8	0.8
11	NAC-PFA50-B	43.2	60.3	65.1	76.9	4.0
12	RAC-PFA50-B	35.5	46.3	46.1	53.1	6.4
13	NAC-PFA50-C	36.2	54.8	72.4	73.9	1.4
14	RAC-PFA50-C	30.1	40.3	53.1	55.0	6.3

*CV = coefficient of variation

In these mixes, it can be seen that SP has played an obvious role in keeping the concrete workable at relatively low water cement ratio (w/c = 0.32), and therefore increased its strength. The impact can be observed on mix NAC-PFA25-A and RAC-PFA25-A in which the strength was 74.5 N/mm^2 and 54.5 N/mm^2 at 28 days respectively. Strengths were satisfactory when compared with the mix design strength (40 N/mm^2) and target mean strength of 53 N/mm^2.

Superplasticized NAC and RAC concretes with a PFA sand substitution level of 25% have achieved the highest compressive strengths at all ages. At this level, strengths of the mix NAC-PFA25-A of 72.1 and 85.8 N/mm^2 were measured at 28 and 90 days respectively; marginally below counterpart mix (NAC-PFA0-A) by 3%, and better than NAC control mix by 5%, but enhanced at 90 days by 20% compared with the control mix. For the similar RAC mix (NAC-PFA25-A), 56.1 and 65.7 N/mm^2 were achieved at 28 and 90 days respectively; the strength was better (by 20% at 90 days) compared with RAC-CM concrete at 28 days. However, the effect of lowered w/c, pozzolanic reaction and filling effect of PFA in addition to the high quality natural granite (used for NAC concrete) played significant roles to achieve these strengths.

For all mixes, and at all PFA percentages, results in general showed RAC concrete strength tended (as was expected) to be lower than that of the NAC; the compressive strength of the RAC control mix was below that of the NAC control mix. The strength of RAC concrete (RAC-PFA25-A) with 25% fine aggregate replacement was also 18, 10, and 4% less than that of the NAC reference mix (NAC-CM) at 28, 56, and 90 days respectively; however, when the strength achieved by RAC-PFA25-A at 90 days is compared with the strength of NAC control mix (NAC-CM) at 28 days, the difference in strengths was reduced by a significant extent (4%). The highest RAC strength was attained by this mix; 56.1, 61.8, and 65.7 N/mm^2 at 28, 56, and 90 days respectively.

The trend of development of compressive strength of the superplasticized RAC mixes is also shown in Fig. 6.1. In general, the development of compressive strengths of RAC concrete have shown similar trend to the NAC standard mix, though the slope of the strength plots of the PFA mixes is slightly greater than mixes without PFA beyond 28 days curing.

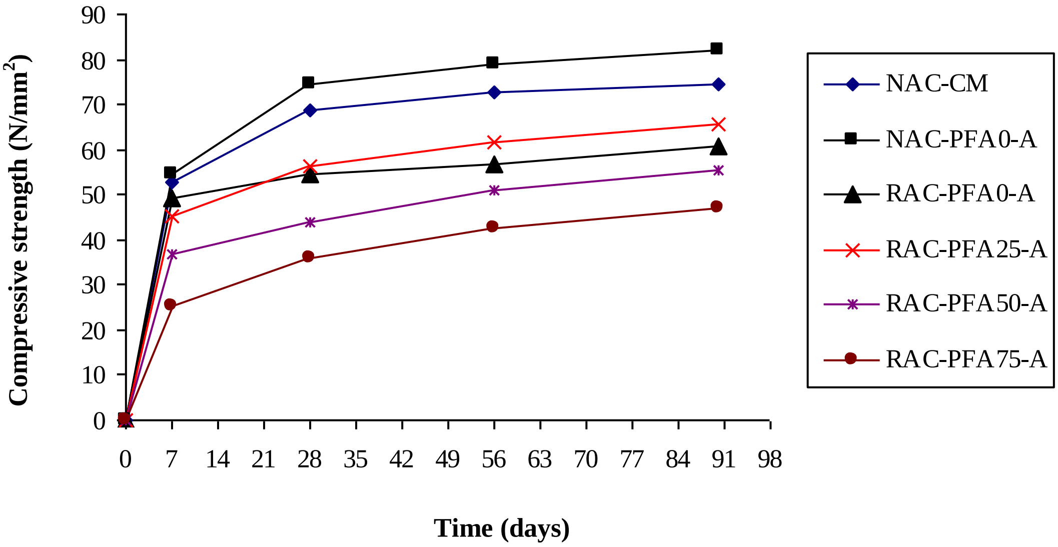


Fig. 6.1a Development of compressive strength of RAC with PFA replacing fine aggregate at different levels (SP type A used) compared to the control mix

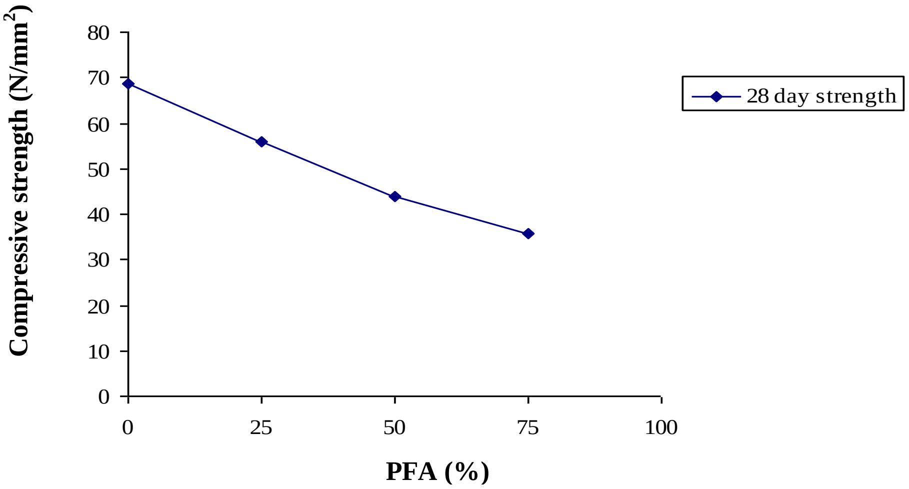


Fig. 6.1b Variation of the 28 day compressive strength of RAC with PFA content

The inspection of the relative strengths in Table 6.8 show the compressive strength at 90 days relative to 28 days; the highest strength ratio (F_{cu90}/ F_{cu28}) for NAC concrete mixes with SP A was 127% attained by NAC-PFA75-A, and similarly RAC-PFA75-A reached 131%; although it was a dry mix. Mixes with SP type C have achieved higher ratios.

Table 6.8 Relative compressive strengths

Mix	Code	F_{cu28} (N/mm^2)	F_{cu90} (N/mm^2	F_{cu28}/ $F_{cu\ control}$ (%)	F_{cu90} / F_{cu28} (%)
1	NAC- (Control mix)	68.5	74.7	100	109
2	RAC- (Control mix)	52.0	53.6	76.0	103
3	NAC-PFA0-A	74.5	82.2	109	110
4	RAC-PFA0-A	54.5	60.8	79.6	112
5	NAC-PFA25-A	72.1	85.8	105	119
6	RAC-PFA25-A	56.1	65.7	82.0	117
7	NAC-PFA50-A	62.5	78.2	91.0	125
8	RAC-PFA50-A	43.9	55.2	64.0	126
9	NAC-PFA75-A	50.7	64.5	74.0	127
10	RAC-PFA75-A	35.8	46.8	52.0	131
11	NAC-PFA50-B	60.3	76.9	88.0	128
12	RAC-PFA50-B	46.3	53.1	68.0	115
13	NAC-PFA50-C	54.8	73.9	80.0	135
14	RAC-PFA50-C	40.3	55.0	59.0	136

Data presented in Table 6.8 are redisplayed in graphical form in Fig. 6.2.

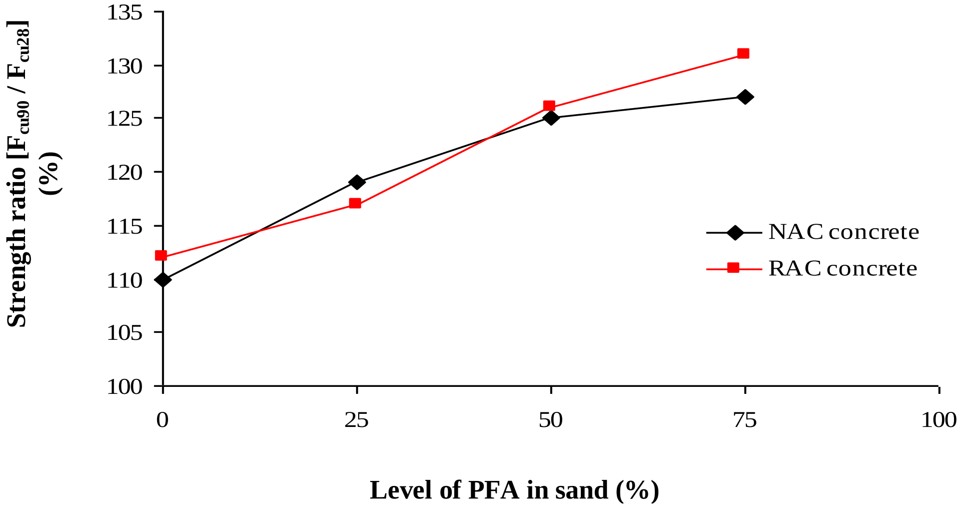


Fig. 6.2 Influence of PFA level in sand on compressive strength ratio

Fig. 6.2 shows the relationship between the PFA level substituted for fine aggregate and the strength ratio F_{cu90}/F_{cu28}; the ratio is clearly increasing with the PFA content for both types of concrete, this however is not necessarily implying that strength is increasing to the same trend; it simply confirms the well documented phenomenon inherent to PFA concrete due to its pozzolanic nature; that is the increase of strength occurs gradually over time.

Data given in Table 6.7 are arranged to show a comparison between early and later age strengths *i.e.* 7 and 90 days as shown in Fig. 6.3. However, concrete strength with PFA needs to be monitored over longer periods to observe the pozzolanic influence of PFA. The dashed line is the line of the characteristic strength (CS line) *i.e.* 40 N/mm^2.

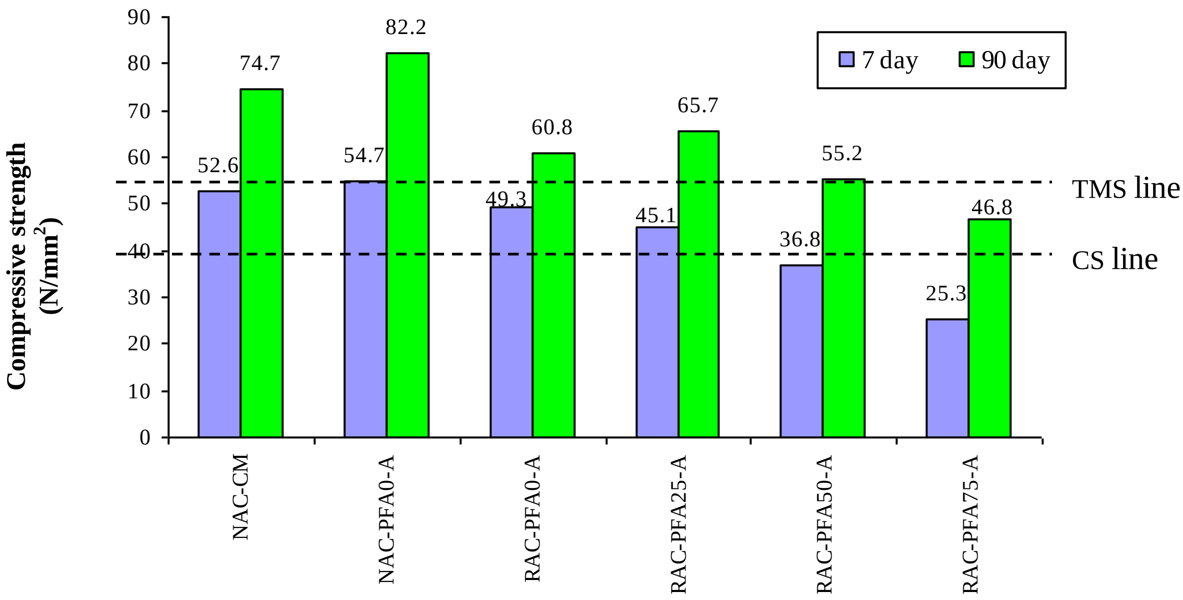


Fig. 6.3 Compressive strength of RAC concrete produced with different PFA levels and SP type A at early (7 days) and later age (90 days)

The influence of superplasticizer type on the compressive strength of RAC concrete mixes containing equal PFA replacement doses is displayed in Fig. 6.4.

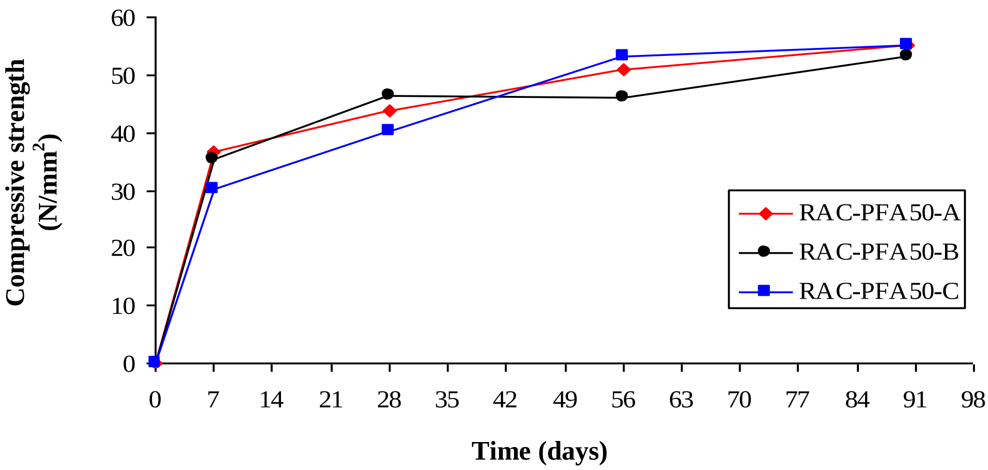


Fig. 6.4 Compressive strength of RAC concretes produced with 50% PFA and different SP types compared with the control mix

Fig. 6.4 shows that all SPs had similar influence on the compressive strength of RAC concrete when a fixed amount of PFA replaced fine aggregate (at 50% PFA content).

Strengths were almost equal after 90 days curing. Thus it can be said that as far as the strength is concerned, all SPs can be considered compatible with the binder (cement and PFA) as they produced comparable strengths. To show a comparison between the compressive strength of NAC and RAC concrete mixes made with SP type A as well as the relationship of strengths with the level of PFA sand replacement in the mixtures, data in Table 6.7 are plotted in histogram form in Fig. 6.5 and as a line diagram in Fig. 6.6.

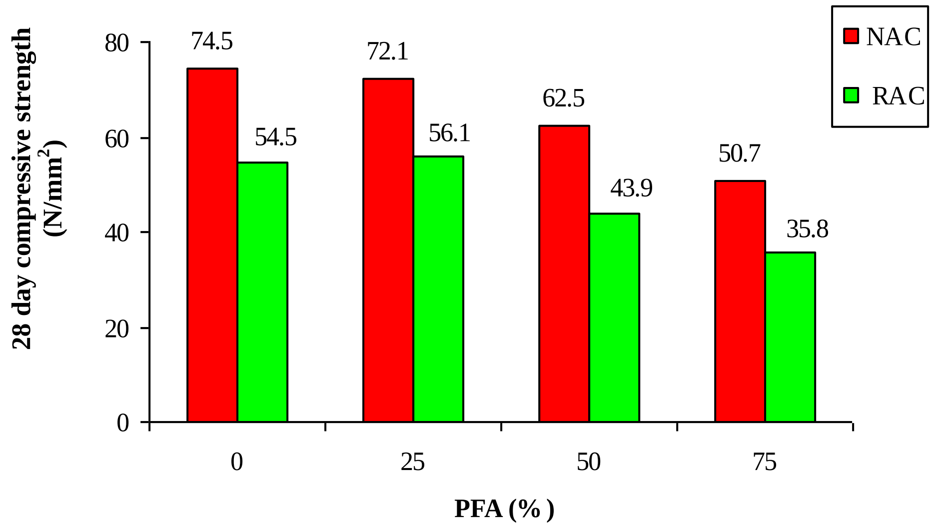


Fig. 6.5 The 28 day compressive strengths of concrete produced with different PFA contents (SP A used)

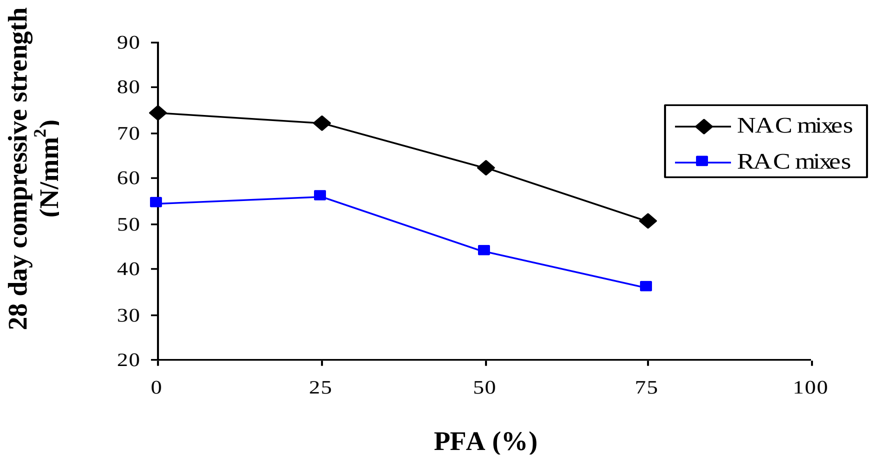


Fig. 6.6 Influence of PFA content on the compressive strength of concrete

It can be seen from Figs 6.5 and 6.6 that as the percentage of PFA replacing fine aggregate increases, the compressive strengths of NAC and RAC decrease. This result is in agreement with many previous studies (more details are given in Chapter 3, Section 3.9 (d) ii which show that PFA concrete's strength perhaps increases up to a certain level of PFA substitution beyond which it usually declines. However, it should also be remembered that this result is not in agreement with other previous research; for instance (Rafat 2004) showed strength increasing despite the increase of PFA substitution level up to 50% sand replacement. Differences in previous findings could be down to differences in physical and chemical properties of PFA used and local constituents of concrete.

When the measured compressive strength of RAC mixes was compared with the mix design strength values as shown in Fig. 6.7, it can be concluded that all NAC and RAC mixes achieved their design characteristic value (40 N/mm^2), but only mixes with 0 and 25% PFA had achieved their design target mean strength (53 N/mm^2). RAC with 25% PFA had achieved greater strength than its control mix by about 3% (56.1 N/mm^2 > 54.4 N/mm^2).

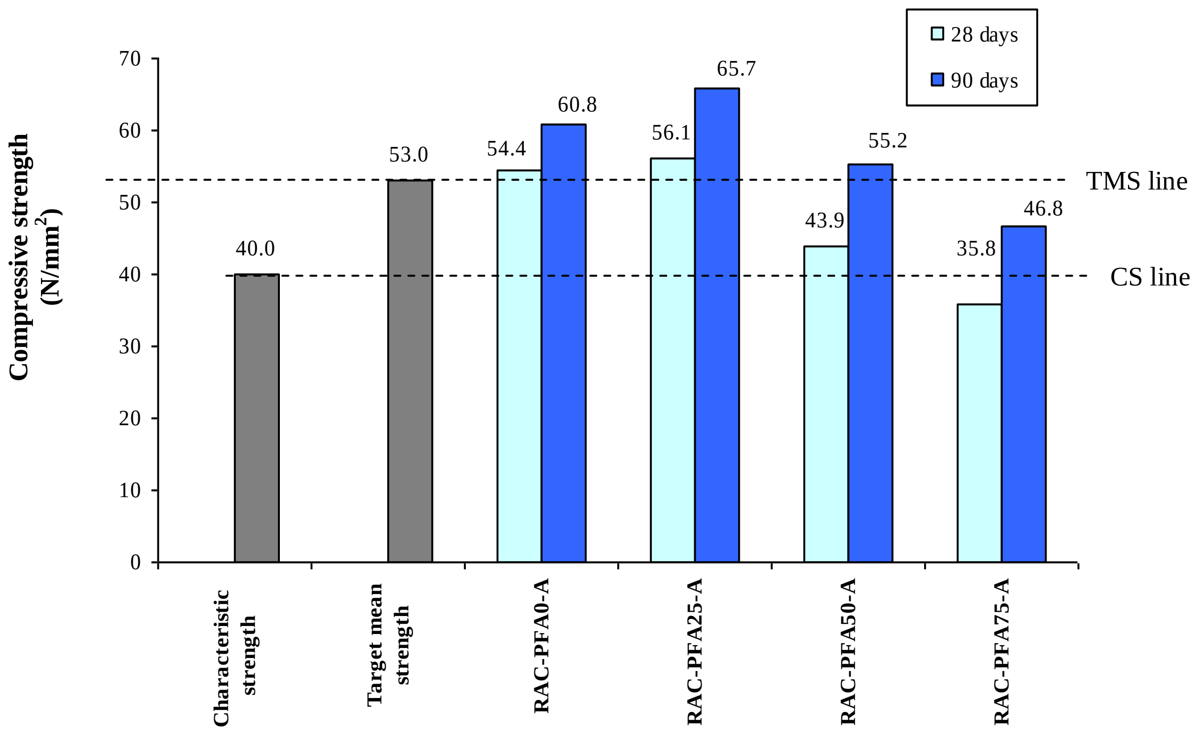


Fig. 6.7 Compressive strength of RAC concrete as compared with mix design values

For most NAC and RAC samples tested, the fracture surface propagated across the paste and crossed the coarse aggregate particles to make a relatively smooth surface as shown in Fig. 6.8. A big bang was heard upon failure in some cases; particularly in the case of higher strength NAC concrete mixes. The bangs were less loud with RAC samples indicating reduced brittleness. Fracture angles often between 30º and 60º to the central vertical axis as shown in Fig. 6.8 were observed.

30º ≤ α ≤ 60º

Fig. 6.8 The shape of crushed RAC and NAC concrete samples

On the other hand, in all cases, the strength of concrete is reduced when PFA replaces fine aggregate, particularly for higher PFA replacement; the physical reason behind that is believed to be the dilution effect of PFA on cement, particularly for high PFA contents. At lower PFA replacement levels, PFA transforms calcium hydroxide in cement or that liberated from cement during hydration to calcium silicate hydrate gel (C-S-H), but as the total PFA content increased, the reaction product (C-S-H) decreased when the available calcium hydroxide is consumed, and the large pozzolan content in the binding system results in extra smaller sized fractions (as compared with the size of cement grains), which tends to disperse the cement grains so that the binding system is diluted resulting in decreased strength beyond 25% PFA replacement. Although in all cases, strength was shown to be modified by the pozzolanic effect of PFA at later ages. In other words, at a higher level of PFA replacement, there would be insufficient Portland cement to produce the required quantities of calcium hydroxide for the secondary reactions [$Ca(OH)_2$ / PFA reaction].

Results showed that RAC exhibited good fresh properties and comparatively high strength, particularly at later ages. Despite the quality of the RA in comparison with the granite NA used in this study, and the reduction of cement content by 25%, RAC performance characteristics remained satisfactory. The best result was obtained with 25% PFA replacement; the strength of RAC concrete was about 56, 62 and 66 N/mm^2 at 28, 56 and 90 days respectively, with a slump of 45 mm. Such performance would not be achieved without the aid of SP. Better results are expected at later ages as the PFA reaction takes longer. Moreover, higher strength would have been obtained if the full amount of cement recommended by the mix design was used.

When SP types B and C were used, less NAC concrete strength was achieved compared to similar mixes with SP type A; the 28 day compressive strength of concrete with 50% PFA was 54.8, 60.3 and 62.5 N/mm^2 when SP types C, B, and A respectively were used; RAC concrete showed similar behaviour. This result supports the author’s previous selection based on the setting behaviour and water demand of the binder paste and the loss of slump of concrete in Chapter 4, to choose SP type A for further mixes.

(iii) Splitting tensile strength

Results of the 28 day tensile splitting strength of NAC and RAC samples are determined using Eqn 5.1 and listed in Table 6.9.

Table 6.9 Tensile splitting strength of NAC and RAC concrete in which PFA replaces fine aggregates at different levels

Mix	Code	PFA (%)	SP type	SP (%)	28 day tensile strength (N/mm^2)	CV* (%)
1	NAC- (Control mix)	0	-	0	4.22	0.86
2	RAC- (Control mix)	0	-	0	3.61	1.51
3	NAC-PFA0-A	0	A	0.8	5.23	1.40
4	RAC-PFA0-A	0	A	0.8	4.00	2.25
5	NAC-PFA25-A	25	A	0.8	5.12	0.85
6	RAC-PFA25-A	25	A	0.8	3.65	1.60
7	NAC-PFA50-A	50	A	0.8	3.17	2.10
8	RAC-PFA50-A	50	A	0.8	2.21	4.31
9	NAC-PFA75-A	75	A	0.8	3.12	0.23
10	RAC-PFA75-A	75	A	0.8	2.00	5.10
11	NAC-PFA50-B	50	B	0.6	3.87	0.61
12	RAC-PFA50-B	50	B	0.6	2.78	1.46
13	NAC-PFA50-C	50	C	2.5	3.76	0.21
14	RAC-PFA50-C	50	C	2.5	2.96	0.86

*CV = coefficient of variation

Tensile strength data for NAC and RAC concrete mixes produced with PFA and SP type A in Table 6.9 are shown in graphical form in Fig. 6.9. It can be seen that tensile strengths were following a similar trend to compressive strengths; results show that beyond 25% PFA sand replacement, the tensile strengths of NAC and RAC were generally decreasing. With 25% replacement level, and perhaps less, the strengths at 28 days were better than the reference mixes. RAC strength was lower than similar NAC in all cases.

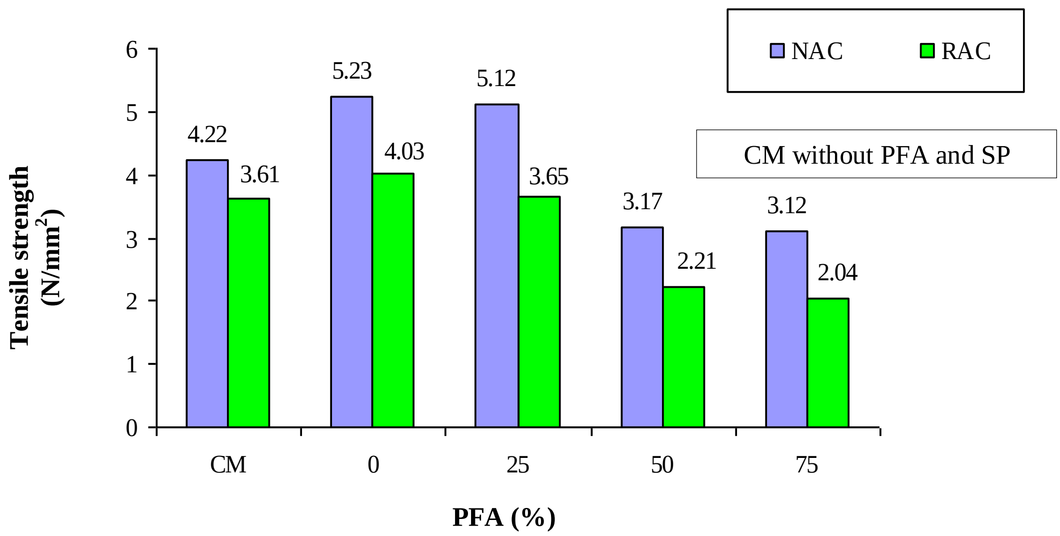


Fig. 6.9 The 28 day tensile strengths produced with different levels of PFA and SP A

Again, the highest possible strength was obtained with 25% PFA replacement for both RAC and NAC concrete; the tensile strength of the mix RAC-PFA25-A was 30% less than the counterpart NAC-PFA0-A, 9.5% less than RAC-PFA0-A, 13.5% less than NAC-CM, and 1.1% better than RAC-CM.

A set of previous key research results in which PFA was used to partially replace fine aggregate were extracted from available literature and arranged in Appendix 3 Table A.3.2; comparison can be made with the author's results listed in this table.

(c) Density of hardened concrete

Density measurements for the concrete mixes are given in Table 6.10.

Table 6.10 Bulk density of hardened concrete with PFA

Mix	Code	SP type	Density of concrete samples (kg/m^3) measured after a period of:			
			7 day	28 day	56 day	CV at 28 days (%)
1	NAC- (Control mix)	A	2441	2471	2466	0.45
2	RAC- (Control mix)	A	2491	2495	2487	1.30
3	NAC-PFA0-A	A	2472	2547	2465	0.25
4	RAC-PFA0-A	A	2498	2569	2480	0.30
5	NAC-PFA25-A	A	2472	2513	2259	6.00
6	RAC-PFA25-A	A	2362	2375	2326	0.50
7	NAC-PFA50-A	A	2305	2334	2309	0.54
8	RAC-PFA50-A	A	2335	2319	2319	0.85
9	NAC-PFA75-A	A	2232	2259	2236	0.40
10	RAC-PFA75-A	A	2227	2293	2279	0.68
11	NAC-PFA50-B	B	2272	2339	2266	0.32
12	RAC-PFA50-B	B	2292	2337	2292	0.94
13	NAC-PFA50-C	C	2297	2323	2291	0.90
14	RAC-PFA50-C	C	2336	2337	2312	1.42

Density results are an average of three samples.

Results showed that the average densities of concrete for all mixes were decreased as the percentage of PFA increased regardless of the type of aggregate. That may be because PFA is lighter than the replaced fine aggregate; however, this can be advantageous for concrete members. Producing concrete of low density implies lighter sections can be used and consequently the size can be reduced representing a financial saving (Khalaf & DeVenny 2004). The density decrease between 28 and 56 days appears anomalous. The trend was consistent but the author remains unable to explain why and recommends this for future research.

Comparisons of the average densities of concrete with an equal amount of PFA (50% PFA 50% fine aggregate) at different ages show that the difference in densities of NAC and RAC concrete were, generally, too small. The average densities at 7 days were 2291 kg/m^3 for NAC and 2321 kg/m^3 for RAC (1.3% more); at 28 days were 2332 kg/m^3 for NAC and 2331 kg/m^3 for RAC (almost the same).

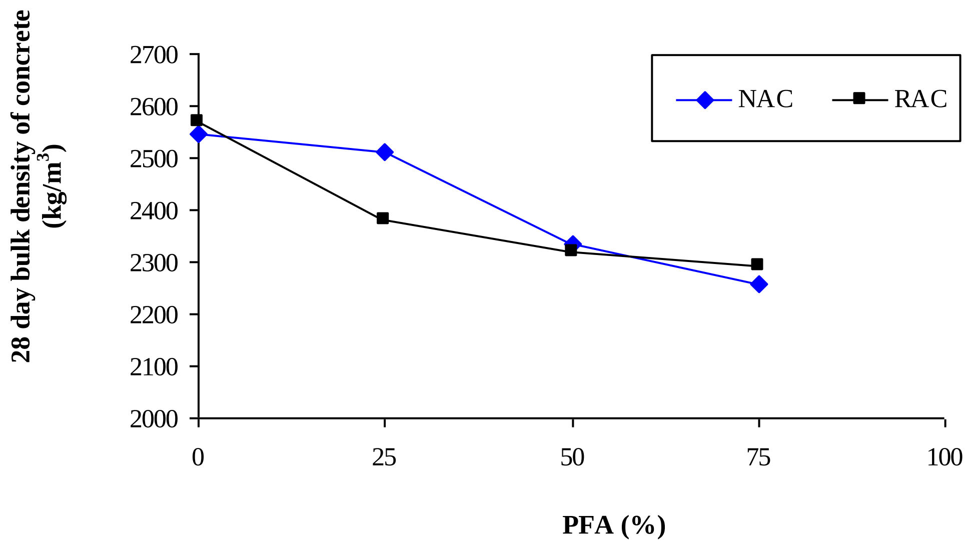


Fig. 6.10 Influence of PFA content on the density of concrete

Fig. 6.10 displays the effect of PFA sand substitution on the hardened density of concrete made with SP type A. At low PFA replacement, the density reduction was marginal; particularly with NAC mixes, but the reduction rose significantly at higher replacement levels, leading to reduced dead load in any finished member.

6.3 SUMMARY

In conclusion, the SP has been shown to play an important role in keeping the PFA concrete workable at relatively low water cement ratios. This however, is being supplemented by the effect created by the spherical shape of PFA grains; spheres move more easily than angular or irregular-shaped grains, however, for high-volume PFA replacement it seemed that the water content needed to be increased for constant SP. Even for an increased SP content, lower constant water content is not likely to be suitable for high volume PFA concrete; enough water is essential for the SP to work effectively. In other words, the increase of SP dosage to the concrete including high PFA content at constant water content will not be enough to address the problem of workability reduction. A compromised design, in which mixes are designed to have relatively higher w/c ratio and workability enhancing SP (or more), is therefore recommended for RAC concrete containing high-volume PFA. This is deemed possible as high-volume PFA mixes are appropriate for concretes requiring low strength.

Results showed that RAC was successfully produced either with the aid of the new generation polymer-based superplasticizer or with the combined influence of SP and PFA, even though 100% RA was used. Up to 25% of fine aggregates may be replaced by PFA without significantly affecting concrete strength. For a mix designed to achieve a compressive strength of 40 N/mm^2 at 28 days; RAC with 25% fine aggregate replacement attained compressive strengths up to 45, 56, 62, and 66 N/mm^2 after 7, 28, 56 and 90 days respectively, despite reducing the cement content by 25%. Comparable results for the tensile splitting strengths were obtained. However, compressive and tensile strengths beyond this level tend to decrease with the increase of replacement level.

Results of the key published data in which fine aggregate was replaced by fly ash at different content and the authors results (this study) are arranged in Table A.3.2; from which rough comparisons can be made, as materials properties and design strengths are different. From the experimental work carried out for this part of the thesis, the following conclusions can be drawn.

1. When PFA replaces fine aggregate in RAC concrete mixes, fine aggregate content is reduced and consequently, excess water available within the aggregate voids decreases. That may be developed by the relatively higher absorption capacity of RA and because the grain size of PFA is well below that of fine aggregate particles. Therefore, results showed that slump decreased as the level of PFA increased: the mix become less cohesive and drier. The spherical nature of PFA grains was likely to have its full effect when enough water (lubricant) is available in the mix, therefore was less advantageous for workability of low w/b ratios and high fly ash content concretes. Higher w/c and proper selection of SP are recommended for high-volume PFA mixes.

2. Most previous knowledge of concrete incorporating PFA was gained based on the characteristics of NAC mixes. This research contributes to clarify the situation when PFA is used for RAC concrete.

3. For superplasticized RAC concrete mixes made with 100% RA, in which PFA replaces fine aggregate, the strengths beyond 25% PFA replacement decreased as the level of PFA increased.

4. Compressive strengths of RAC concrete reached 56 N/mm^2 after 28 days, and achieved higher strength with age reaching 66 N/mm^2 after 90 days. These were achieved for a mix designed to attain 40 N/mm^2 after 28 days curing, although the cement content was reduced by 25% thus contributing to its medium strength (42.5 N/mm^2 after 28 days). Such strength levels, together with the well known benefits of PFA to the long-term performance of concrete, make this type of concrete more economical and satisfactory for many structural applications.

5. Strengths of concrete containing PFA increased over time, regardless of aggregate type. Therefore later age strengths of the resultant RAC are likely to increase.

6. PFA could be used as a substitute for fine aggregate as it improves the quality of RAC concrete; a limit of 20-25% PFA fine aggregate replacement for superplasticized mixes is recommended. Although, of course, higher PFA content is possible, particularly for low strength concrete mixes. The achieved strengths of concrete with up to 50% replacement are good enough for many ordinary uses.

7. Better quality RAC is anticipated when a combination of RA and good quality NA (*i.e.* blended aggregate) is used.

CHAPTER 7

RAC PRODUCED BY CEMENT REPLACEMENT WITH PFA AND RGD

7.1 INTRODUCTION

Concrete is still the predominant building material all over the world. It is always necessary to identify less expensive cement substitutes. Cement is the most costly and energy-intensive component of concrete and the unit cost can be reduced by partially replacing cement with fly ash (Rafat 2003a). It has been shown (Chapter 3) that PFA was beneficial to NAC concrete in several ways. The aim of the research presented here was to evaluate the possibility of using PFA complying with BS EN 450: 2005 as a partial cement substitute for RAC concrete.

It is well known that increasing the fines content, particularly for medium to high range cement contents, results in increased water demand. In the absence of enough water, the available free water and cement tend to work less effectively. This situation leads to the agglomeration of cement particles which makes the mix stiffer. The additional water for wetting the finer fly ash definitely results in reduced strength. In such cases, it may be possible to avoid the lack of workability and decline of strength by simply increasing cement content. An alternative to this lies in reducing the mixing water content and use of superplasticizing admixtures (Dhir *et al.* 1998). Therefore, the second technique *i.e.* the reduction of mixing water and the use of SP to keep the workability will be adopted in this research, as the increase of cement will result in increased cost.

It is well known that for concretes with compressive strength below 40 N/mm^2, the coarse aggregate rarely becomes strength-limiting as a relatively high water to cement ratio is often used *i.e.* within the range of 0.5 - 0.7. Within this range, the hardened paste and the interfacial zone between the paste and the aggregates are the weakest link in the concrete system (Beshr *et al.* 2003). Similarly for such concrete, the mineralogy of coarse aggregates is not a major factor in the mix design unless it is proven to contain a sufficient amount of minerals harmful to concrete (Neville 2003).

In contrast, literature shows that with higher strength concrete, for which a lower w/c ratio is frequently used (*i.e.* w/c < 0.45), the hardened paste and interfacial zone are strong enough and do not limit the strength; it is the strength of coarse aggregate and maybe its mineralogy that control the concrete's strength.

Durability is the ability to withstand external aggressive environmental factors during the designed working life of the structure, and is very much related to the permeability and compressive strength of concrete. It has been widely reported that most aspects of concrete durability are enhanced through the use of fly ash (McCarthy *et al.* 1999). In general, well compacted, homogenous, cohesive and adequate strength concrete is usually durable. Therefore, the basic mechanical properties were investigated at this stage, whereas durability aspects of RAC itself represent issues for another research study and, as such, are not included here.

Similar to mixes in Chapters 5 and 6, all RAC concretes in Chapter 7 were also created using 100 % RA instead of NA. PFA and RGD were used to partially substitute cement.

7.2 RAC CONCRETE PRODUCED WITH PFA PARTIALLY REPLACING CEMENT

a) General

It is worth reminding the reader that the two major earlier findings are: firstly, SP type A was selected based on the study of setting times and water demand in Chapter 4, as well as strength in Chapter 5, and secondly the best combinations found for SP type A and water reduction (WR), for the materials used, were 12% WR for 0.6% SP, 16% WR for 0.8% SP and 25% WR for 1.2% SP (see Section 4.8 (e) and Table 4.27 for more details). Therefore, these findings will be adopted in producing the concrete mixes in this part of the investigation.

However, as the above mentioned findings resulted in three possible combinations of SP and WR as mixing options, the question now is that out of these options which is the best? Which one produces the most favourable fresh and hardened properties of concrete? Mixes in the following section were designed to provide the answer. Once the best option is selected, RAC concrete mixes satisfying the aim of this part of the study can then proceed.

(b) Selection of the best mixing option

To identify the best mixing option on the basis of fresh and hardened properties of concrete, eight mixes were produced (Mixes 1 to 8) with no PFA. These mixes comprise two standard mixes (Mixes 1 and 2), and six other mixes (Mixes 3 to 8) with the three aforesaid mixing options. All these were tried with both NA and RA as shown in Table 7.1.

Table 7.1 Concrete mixes without PFA (Mixes 1 to 8)

Mix	Code	SP (%)	WR (%)	PFA (%)
1	NAC-CM	0	0	0
2	RAC-CM	0	0	0
3	NAC-12WR-0.6SP	0.6	12	0
4	RAC-12WR-0.6SP	0.6	12	0
5	NAC-16WR-0.8SP	0.8	16	0
6	RAC-16WR-0.8SP	0.8	16	0
7	NAC-25WR-1.2SP	1.2	25	0
8	RAC-25WR-1.2SP	1.2	25	0

Based on these results, further mixes incorporating PFA will be produced (Mixes 9 to 12) to investigate the influence of partial replacement of cement by PFA on the strength of superplasticized RAC mixes.

The two standard or reference mixes were designed to achieve a characteristic compressive strength of 50 N/mm^2 after 28 days with a slump of 10-30 mm and Vebe time of 6-12 s for NAC and RAC mixes in accordance with the Building Research Establishment mix design method (BRE 1992).

Table 7.2 Mix proportion of standard mixes (Mixes 1 and 2)

Material	Mass (kg/m^3)	
	NA	RA
Cement	425	425
Water	170	170
Coarse aggregate	1280	1235
Fine aggregate	545	530
Wet density	2420	2360

The detailed mix design sheets are shown in Appendix 2. All specimens were cast in steel cubes 100 × 100 × 100 mm, and compacted in a vibrating table according to BS 1881-108: 1983. All specimens were stripped after 24 hours and cured in water tank at a temperature of 21°C. Samples were tested for density and compressive strength after 28, 56, and 90 days to meet the requirements of BS 1881-114: 1983 and BS EN 12390-3: 2002 respectively. Cubes were also tested for tensile splitting strength after 28 days in accordance with BS 1881-117: 1983. Three samples were tested in all cases and average results obtained.

(i) Workability of no PFA mixes

The workability of mixes was measured in accordance with the relevant standard previously mentioned in Chapter 5; workability is used here in addition to strength to aid the selection process. The results are summarised in Table 7.3 and discussed in Section 7.2 (b) iii.

Table 7.3 Workability without PFA (Mixes 1 to 8)

Mix No	Code	w/c*	SP (%)	WR (%)	Slump (mm)	Vebe (s)	Visual observations
1	NAC-CM	0.40	0	0	15	12	Uniform mix
2	RAC-CM	0.40	0	0	10	14	Uniform mix
3	NAC-12WR-0.6SP	0.35	0.6	12	20	7	Uniform mix
4	RAC-12WR-0.6SP	0.35	0.6	12	20	9	Uniform mix
5	NAC-16WR-0.8SP	0.34	0.8	16	20	8	Uniform, cohesive mix
6	RAC-16WR-0.8SP	0.34	0.8	16	25	7	Uniform, cohesive mix
7	NAC-25WR-1.2SP	0.30	1.2	25	40	14	Signs of segregation, shear slump
8	RAC-25WR-1.2SP	0.30	1.2	25	25	10	Less cohesive mix

* w/c were different as WR were different, for instance 25% WR will reduce the w/c ratio to $0.40 \times (1 - \frac{25}{100}) = 0.30$.

Discussion of the results will be made later.

(ii) Compressive, tensile, and flexural strength of no PFA mixes

Strength is one of the most important factors controlling the selection of mix components; in addition to workability, strength is a major decisive factor. Strengths were measured in a similar way to that introduced in Chapter 5, and results are given in Table 7.4.

Table 7.4 Compressive strength of mixes without PFA (Mixes 1 to 8)

Mix No	Code	Compressive strength after: (N/mm^2)		
		28 days	56 days	90 days
1	NAC-CM	69.0	71.2	76.0
2	RAC-CM	52.0	58.0	58.3
3	NAC-12WR-0.6SP	73.0	88.8	91.5
4	RAC-12WR-0.6SP	55.0	66.6	68.2
5	NAC-16WR-0.8SP	74.0	89.7	91.7
6	RAC-16WR-0.8SP	64.0	72.0	73.0
7	NAC-25WR-1.2SP	81.3	96.6	97.2
8	RAC-25WR-1.2SP	52.5	67.6	69.3

Due to good quality NA (granite) being used in the mixes, all NAC concretes have produced strength values better than RAC concrete. RA is usually formed of aggregates of different mineralogy, shapes, *etc*., and attains less strength than its counterpart NA. Results are in agreement with several previous research findings (Hansen 1985; Ravindrarajah & Tam 1985; Dhir 2001; Etxeberria *et al.* 2007).

The measured tensile splitting and flexural strengths are displayed in Table 7.5.

Table 7.5 Tensile splitting and flexural strength of mixes without PFA (Mixes 1 to 8)

Mix	Code	Splitting tensile strength after: (N/mm^2)			Flexural strength after: (N/mm^2)	
		28 days	56 days	90 days	28 days	56 days
1	NAC-CM	4.70	3.78	4.60	8.10	9.75
2	RAC-CM	3.60	4.04	3.63	7.75	7.87
3	NAC-12WR-0.6SP	4.92	6.00	-	-	-
4	RAC-12WR-0.6SP	3.80	4.40	-	-	-
5	NAC-16WR-0.8SP	5.78	7.17	7.05	11.3	11.3
6	RAC-16WR-0.8SP	4.79	4.35	4.92	10.7	8.20
7	NAC-25WR-1.2SP	3.28	6.98	-	-	-
8	RAC-25WR-1.2SP	2.43	5.75	-	-	-

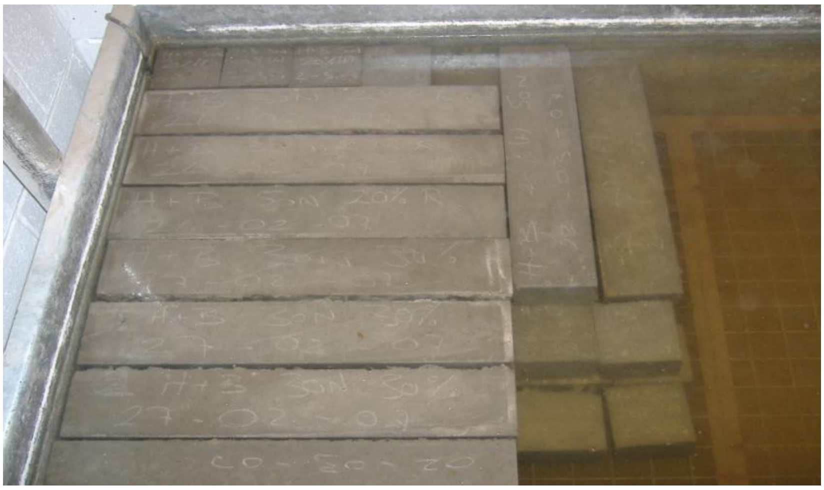


Fig. 7.1 Beams and cubes kept in water tank at 20 ± 2°C

Density of hardened concrete and pulse velocity through concrete cubes were measured before crushing. The density measured was the average of all cubes tested for compression (three cubes) and for tensile splitting strength (three cubes); similarly, pulse velocity was the average of five velocities measured through each cube. Therefore each density listed in Table 7.6 is the mean of six results, and each pulse velocity the average of 30 readings.

Table 7.6 Density and pulse velocity of no PFA concrete (Mixes 1 to 8)

Mix	Code	Density (kg /m^3) and pulse velocity (km/ s) after:		
		28 days	56 days	90 days
1	NAC-CM	2393 (4.85)	2288 (4.76)	2278 (4.86)
2	RAC-CM	2280 (4.45)	2305 (4.51)	2292 (4.50)
3	NAC-12WR-0.6SP	2305 (4.82)	2309 (5.02)	2311 (4.85)
4	RAC-12WR-0.6SP	2334 (4.55)	2346 (4.84)	2305 (4.96)
5	NAC-16WR-0.8SP	2316 (4.85)	2314 (5.04)	2322 (5.1)
6	RAC-16WR-0.8SP	2343 (4.54)	2365 (4.88)	2338 (4.69)
7	NAC-25WR-1.2SP	2320 (4.69)	2302 (5.03)	2312 (4.83)
8	RAC-25WR-1.2SP	2341 (4.37)	2334 (4.68)	2338 (4.72)

Concrete performance, in terms of workability and strength, obtained in the preceding paragraphs will be used for the selection of the best mix. The interpretation of the above results leading to the choice of the preferred mix will be detailed in the following paragraphs.

(iii) Selection of the best mixing option

A comparison has been made between NAC Mixes 3, 5, and 7 as well as between RAC Mixes 2, 4, and 8 in order to select the best mixing option (in other words the best combination of water reduction and SP; the one that exhibits the most favourable fresh and hardened properties of concrete.

Analysing the workability and strength results listed in Tables 7.4 and 7.5 it is evident that for the NAC concrete, Mix 7 (NAC-25WR-1.2SP) with 25% water reduction (the lowest w/c of 0.3) and SP content of 1.2% of cement by mass has achieved the highest value at 28 days (81.3 N/mm^2), but conversely, attained the lowest tensile strength (3.28 N/mm^2). This mix also showed a shear slump indicating lack of cohesion between the concrete components; signs of segregation and bleeding were also observed. Segregation and bleeding usually lead to honeycombing, heterogeneity and less cohesive concrete. However, little may be observed in laboratory concrete cubes; the influence becomes more apparent in larger sections like slabs and mass concrete pours. These properties could ultimately impair the durability of concrete and gradually reduce its performance.

The similar RAC Mix 8 (RAC-25WR-1.2SP) produced the lowest 28 day compressive strength (52.5 N/mm^2), and the lowest tensile strength (2.43 N/mm^2). Again this mix also showed a rapid loss of slump over time. Therefore, the mixing option with 25% WR and 1.2% SP was rejected although it achieved the highest compressive strength of all RAC mixes.

NAC concrete Mixes 3 and 5 with 12% and 16% WR and 0.6% and 0.8% SP respectively produced almost equal compressive strengths which increased at similar rates. Both concrete mixes attained a compressive strength better than that of the reference mix (NAC-CM) at all ages. In the same way both mixes developed tensile splitting strengths better than those exhibited by the reference mix. However, Mix 5 achieved higher tensile strengths at 28 and 56 days than Mix 3 (5.78 N/mm^2 > 4.92 N/mm^2 and 7.17 N/mm^2 > 6.0 N/mm^2); remarkably it also shows better cohesion and more homogeneity than other mixes.

For RAC concrete mixes, Mix 6 was observed to attain the highest compressive and tensile strength at all ages (*i.e.* 28, 56, and 90 days), the compressive strength was in general comparable to the reference mix; it was just 7% less at 28 days, 4% less at 90 days but 1% more at 56 days. Visual inspection of the fresh mixture showed that this mix was uniform and cohesive. The increased strength of RAC concrete is partially due (Salem & Burdette 1998) to its rough texture and the absorption capacity of the adhered old mortar that provides better bonding and interlocking between the cement paste and the recycled aggregate particles themselves.

Based on the aforementioned discussion, concrete mixes with the mixing option of 16% WR and 0.8% SP were chosen to be used for the production of the other mixes in which PFA partially replaces cement: Mixes 9 to 12. Results showed that the compressive strength should not be considered as the only decisive factor affecting the selection of the best concrete mix.

(c) Concrete mixes with PFA replacing cement

(i) General

In this part, four mixes *i.e.* Mixes 9 to 12 were cast. Two (Mixes 9 and 10) were standard mixes designed to achieve 50 N/mm^2 after 28 days curing with a slump of 10-30 mm and Vebe time of 6-12 s for NAC and RAC mixes, in accordance with the

Building Research Establishment's mix design method (BRE 1992) that was especially prepared for fly ash concrete mixes for which the mix design was previously explained in detail.

In the other two mixes (Mixes 11 and 12), 30% of the cement content by mass was partially replaced with PFA. The 30% level was selected based on the findings discussed in Chapter 3; water content was also reduced by 16% combined with 0.8% SP (as proven to be the best combinations for materials at hand in Chapter 4; refer to Table 4.27). The specific surface areas of cement and PFA are nearly the same (290-390 m^2/kg); therefore no significant amount of water would be lost due to difference of surface areas. The relative density of the fly ash particle (2.25) is however less than that of the cement (3.15) therefore there will be a greater volume of paste if substituted weight for weight. The BRE mix design method used in this investigation covers the differences arising, with no need to further adjust the mix. Mixes are as shown in Table 7.7 and mix proportions of standard mixes with 30% PFA in Table 7.8.

Table 7.7 Concrete mixes with PFA (Mixes 9 to 12)

Mix	Code	SP (%)	WR (%)	PFA (%)
9	NAC-30PFA	0	0	30
10	RAC-30PFA	0	0	30
11	NAC-16WR-0.8SP-30PFA	0.8	16	30
12	RAC-16WR-0.8SP-30PFA	0.8	16	30

Table 7.8 Mix proportion mixes contains 30% PFA (Mixes 9 and 10)

Material	Mass (kg/m^3)	
	NA	RA
Cement	345	345
PFA	145	145
Water	155	155
Coarse aggregate	1265	1210
Fine aggregate	540	515
Wet density	2450	2370

Mixes in this part were coded to show whether the mix contained natural aggregate concrete (NAC) or recycled aggregate concrete (RAC) and to show the percentage amount of water reduction (WR) when these mixes were produced, the amount of superplasticizer (SP) and the level of PFA in the mix, for example, Mix 12 is coded as: RAC-16WR-0.8SP-30PFA indicating the use of recycled coarse aggregate in this mix, water was reduced by 16%, an SP content of 0.8% of the binder content, and 30% of the cement replaced by PFA.

Basic mechanical properties of concrete tests were conducted on RAC concrete specimens in compliance with the relevant British and European standards. Tests include compressive strength, tensile splitting strength, flexural strength (modulus of rupture) and modulus of elasticity. The results of these properties were analysed in comparison to the produced similar NAC concrete specimens (control mixes).

(ii) Workability of concrete when PFA replaces cement

Table 7.8 show the workability measurements (standard slump and Vebe time) and visual remarks made on the produced mixes.

Table 7.9 Workability of mixes with PFA replacing cement

Mix	Code	w/c	w/b	SP (%)	WR (%)	Slump (mm)	Vebe (s)	Visual observation
9	NAC-30PFA	0.4	0.32	0	0	25	7	Uniform, cohesive mix
10	RAC-30PFA	0.4	0.32	0	0	20	10	Uniform mix
11	NAC-16WR-0.8SP-30PFA	0.34	0.27	0.8	16	85	2	Uniform, wet but no segregation
12	RAC-16WR-0.8SP-30PFA	0.34	0.27	0.8	16	60	4	Uniform, cohesive mix

Results showed that replacing 30% PFA in concrete leads to an increase in slump particularly with NAC mixes. The combination of PFA and SP influenced the slump significantly; slumps of mixes with PFA and SP increased from 25 to 85 mm and 20 to 60 mm for NAC and RAC concrete respectively, the Vebe time was significantly decreased. That is probably due to the diffusive power of SP, the smaller size and the spherical shape of the PFA particles. It is well known that concrete with PFA replacing cement requires lower SP content compared to other concrete (Vanchai *et al.* 2007). Ball bearing effects in spherical fly ash particles reduce internal friction between aggregates and results in increased workability of fresh concrete. The inspection of hardened PFA concrete showed that, in almost all cases, the external surface of concrete is very fine and smooth compared to the reference mix. Compressive strengths are given in Table 7.10; rejected mixes (Mixes 3, 4, 7, and 8 are excluded).

(iii) Compressive strength of concrete when PFA replaces cement

Table 7.10 summarises the measured compressive strengths of concrete.

Table 7.10 Compressive strength of mixes without and with PFA replacing cement

Mix	Code	PFA (%)	Compressive strength after: (N/mm^2)		
			28 days	56 days	90 days
1	NAC-CM	0	69	71.2	76.0
2	RAC-CM	0	52	58.0	58.3
5	NAC-16WR-0.8SP	0	74	89.7	91.7
6	RAC-16WR-0.8SP	0	64	72.0	73.0
9	NAC-30PFA	30	65	68.5	75.6
10	RAC-30PFA	30	50.8	52.5	59.5
11	NAC-16WR-0.8SP-30PFA	30	75.4	84.5	90.0
12	RAC-16WR-0.8SP-30PFA	30	61.6	67.5	69.0

Note: Mixes 11 and 12 were also tested at 7 days to check the early age strength of concrete with PFA; 50 and 36.4 N/mm^2 were measured.

A comparison between all NAC mixes and their characteristic strength (CS) and target mean strength (TMS) after 28 days has been made and displayed in Fig. 7.2. It is clear from the graph that all NAC mixes have achieved their target strengths.

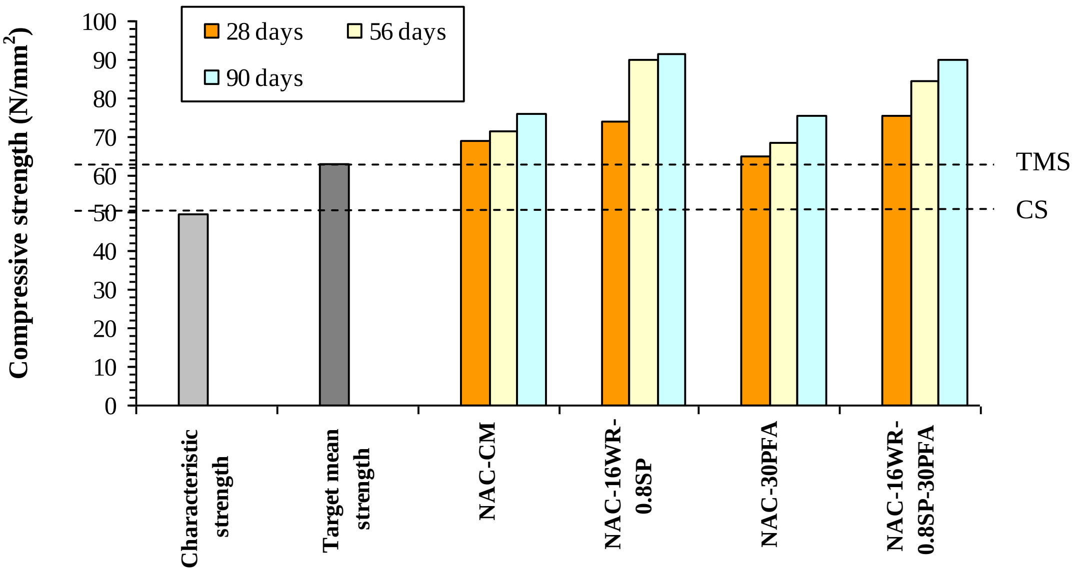


Fig. 7.2 Compressive strength of NAC mixes with PFA at different ages compared to control mixes

In a similar way, the comparison is presented in Fig. 7.3 for RAC concrete mixes; the replacement of 30% of cement content by an equal weight of PFA slightly reduced the compressive strength of RAC concrete at early ages but later improved it with time (compare Mixes 2 and 10; RAC-CM and RAC-30PFA). However, the strength of RAC concrete with PFA and SP was marginally below the target mean strength at 28 days, but was greater beyond 56 days; the strength at 90 days (69 N/mm^2) was equivalent to that of the NAC reference mix at 28 days.

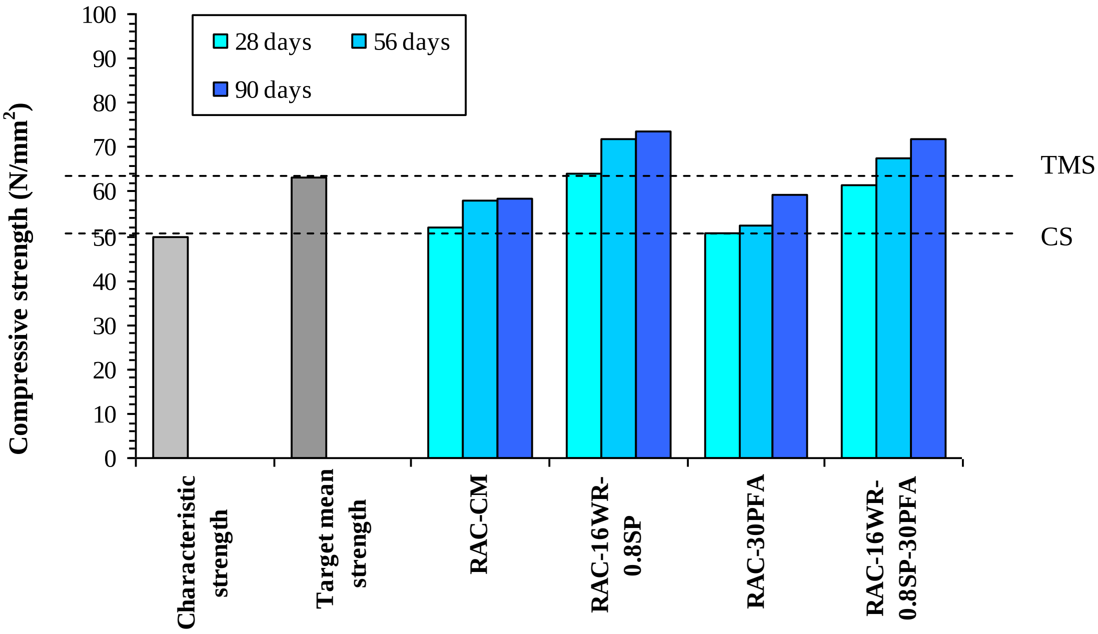


Fig. 7.3 Compressive strength of RAC mixes with PFA at different ages compared to control mixes

A comparison of compressive strengths of concrete mixes (NAC and RAC) with 30% PFA with and without SP is shown in Fig. 7.4.

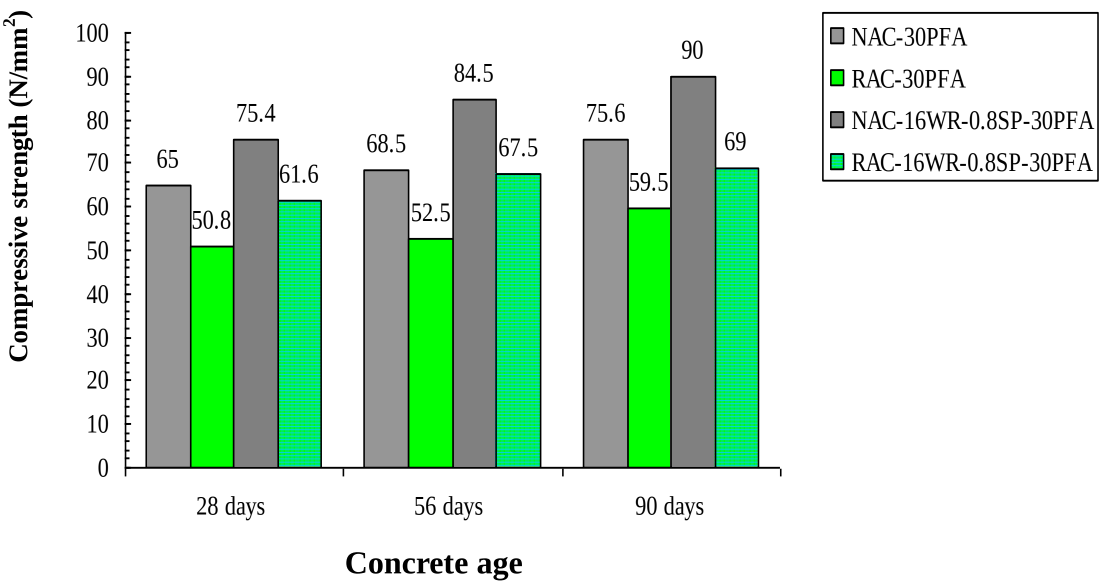


Fig. 7.4 Comparison of compressive strength of concrete with PFA at different ages

The data presented here clearly showed gradual increase of cube strength of concrete with PFA over time; that is due to continued hydration of cement and the pozzolanic nature of PFA. The long term reactivity, although slower, of the fly ash means that there is also potential for higher ultimate strength (Dhir & McCarthy 1999).

A set of previous research results in which PFA was used to partially replace cement in concrete mixes and the author's results are presented in Appendix 3 Table A.3.3; although the concrete components and the design strengths were different and are likely to have different properties; the results were comparable.

(iv) Tensile strength of concrete when PFA replaces cement

It is well known that the tensile and flexural strengths are very much related to the durability of concrete. The resistance to the ingress of harmful substances, which causes damage to concrete and reinforcement corrosion, is much improved as the tensile and flexural strengths are increased. The data for tensile splitting and flexural strengths are shown in Table 7.11.

Table 7.11 Tensile splitting and flexural strength of mixes with PFA replacing cement

Mix	Code	PFA (%)	Splitting tensile strength after: (N/mm^2)			Flexural strength after: (N/mm^2)		$F_{t\,28}/F_{cu\,28}$ (%)
			28 d	56 d	90 d	28 d	56 d	
1	NAC-CM	0	4.7	3.78	4.60	8.10	9.75	15
2	RAC-CM	0	3.60	4.04	3.63	7.75	7.87	14
5	NAC-16WR-0.8SP	0	5.78	7.17	7.05	11.30	11.3	13
6	RAC-16WR-0.8SP	0	4.79	4.35	4.92	10.70	8.20	13
9	NAC-30PFA	30	3.43	4.30	4.82	7.52	7.00	19
10	RAC-30PFA	30	3.80	3.80	3.36	7.16	6.87	13
11	NAC-16WR-0.8SP-30PFA	30	4.20	5.50	4.67	10.6	11.2	18
12	RAC-16WR-0.8SP-30PFA	30	4.12	4.02	4.36	7.40	8.83	15

$F_{t\,28}$ and $F_{cu\,28}$ are 28 day tensile and compressive strengths respectively.

The data presented in Table 7.11 showed RAC-CM offered strengths slightly less than NAC-CM. The other concrete mixes form two categories; superplasticized mixes without PFA, and superplasticized mixes with PFA. The former achieved better strength relative to the reference mixes; the latter lower strengths, although they were more comparable. This simply indicates a mild adverse influence of PFA on tensile and flexural strength. Figs 7.5 and 7.6 show tensile and flexural strengths compared to control mixes.

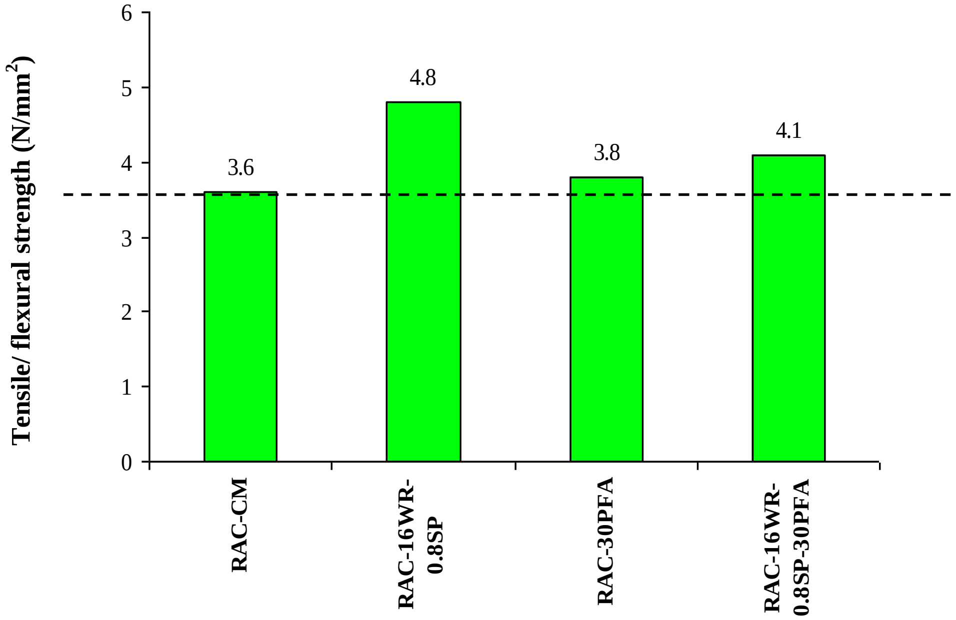


Fig. 7.5 Tensile strength of RAC with and without PFA compared to control mixes

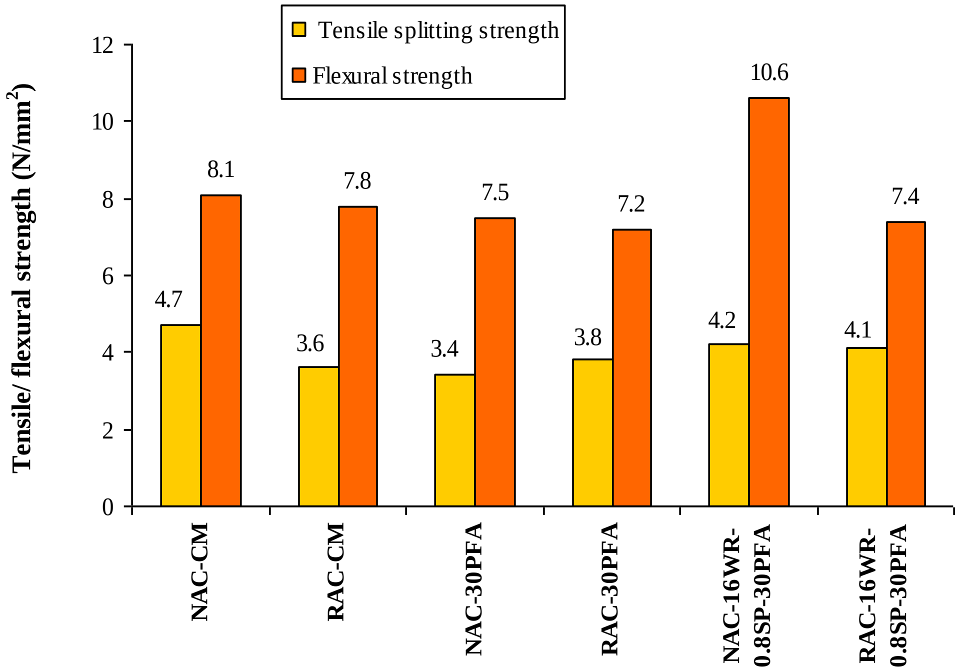


Fig. 7.6 Tensile splitting and flexural strength of mixes (NAC and RAC) with PFA compared to control mixes

Fracture surfaces of concrete samples are shown in Fig. 7.7.

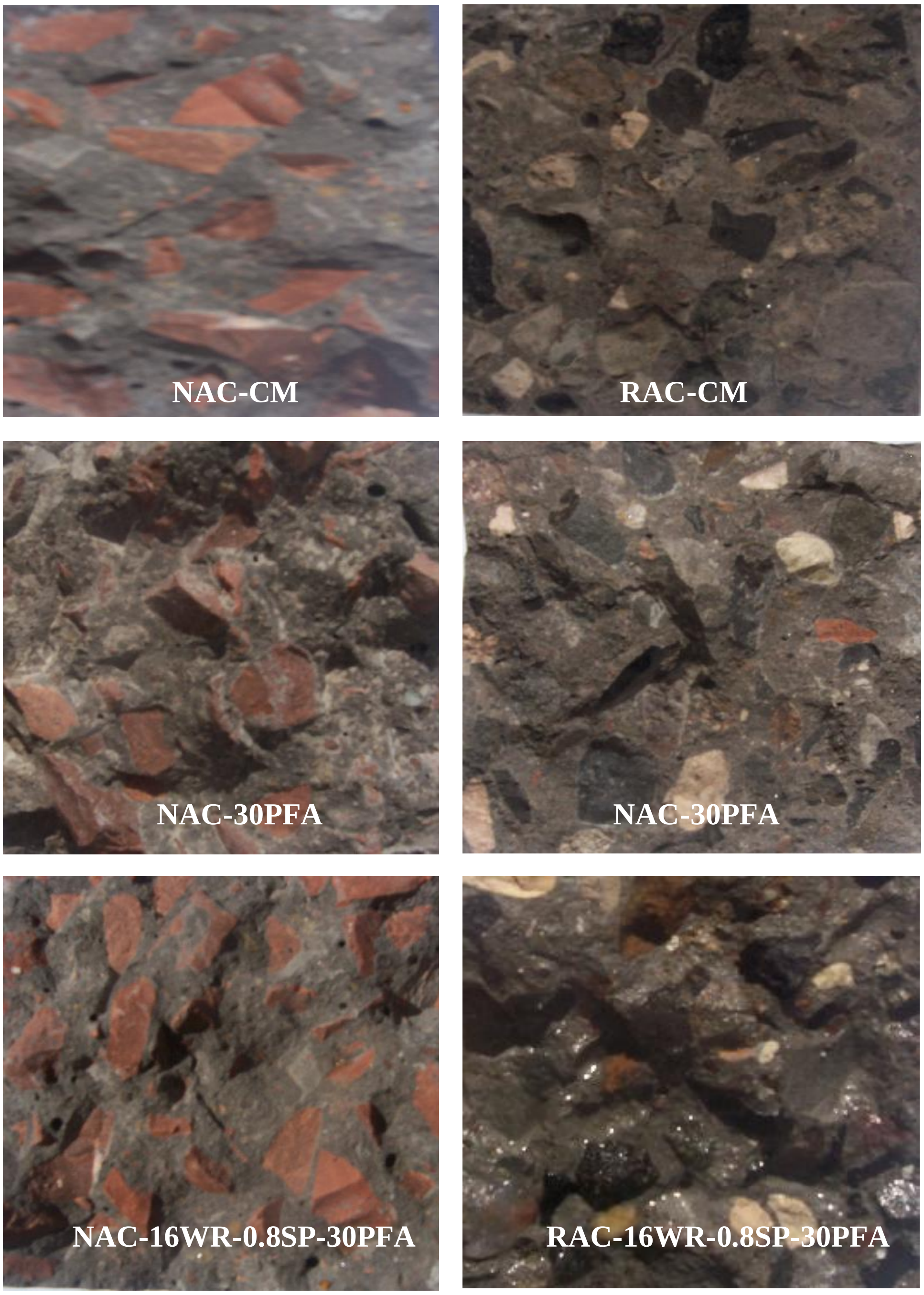


Fig. 7.7 Fracture surfaces of concrete samples

Close inspection of the crushed samples in Fig 7.7 showed less bond failure on the surface of the mixes, particularly in the superplasticized case (failure occurs mainly throughout the RA particles) leading to say that the improved tensile strength was a result of improved bond and interlocking of the aggregate and the concrete matrix. Many previous studies (Tavakoli & Soroushian 1996; Sague *et al.* 2001; Etxeberria *et al.* 2007) in which recycled aggregates were used, but with no PFA, showed that RAC produces tensile splitting strength higher than that obtained using natural aggregates.

However, results of the author's investigation showed that RAC concrete, in which PFA was used, achieved comparable results.

In conclusion, SP has been shown to play a significant role in maintaining good workability of fresh concrete and increasing the compressive strength of concrete mixes as reflected in the higher results obtained *e.g.* the strength of Mix 6 (RAC-16WR-0.8SP; a mix without PFA) was 64 N/mm^2, 23% higher than the RAC reference mix at 28 days.

With reference to compressive strength, RAC mixes with PFA and SP have also produced comparable results. For RAC mix with PFA and SP Mix 12 (RAC-16WR-30PFA-0.8SP), the compressive strength was 18.5% higher than the reference mix. However, earlier research (Chapter 3) has shown that strengths of concretes with PFA tend to increase over longer time than conventional Portland cement concrete; the PFA pozzolanic reaction is known to be relatively slow. Results were in agreement with previous research; a sample of six cubes of Mixes 11 (NAC-16WR-0.8SP-30PFA) and 12 (NAC-16WR-0.8SP-30PFA) were tested at 7 days resulting in a compressive strength of 50 and 36.4 N/mm^2 while attaining an average strength of 90 and 69 N/mm^2 after 90 days (increased by 80% and more than 89% respectively) indicating substantial strength increase. Therefore the ultimate strengths at later ages beyond 90 days are expected to increase. All mixes with PFA and SP or with only SP has achieved their target mean strength; this simply indicated that SP is an essential component in producing good quality RAC incorporating PFA.

Test results indicated that the use of PFA with the aid of SP was capable of producing comparatively strong RAC, particularly at later ages. RAC strengths varied from 50-64 N/mm^2 at 28 days, 52-67 N/mm^2 after 56 days, and 58-73 N/mm^2 after 90 days were achieved. Commercial high-strength concrete mixtures are often designed to obtain 50-80 N/mm^2 compressive strength at 28 days (Mehta 2002). Comparing with commercial concrete, the achieved strengths were good enough to use RAC incorporating PFA for a variety of structural applications.

Tensile strengths at 28 days were also comparable; the relative strength ranged between 76.5 to 102% of the NAC reference mix (Mix 1; NAC-CM), however all RAC mixes were shown to achieve a tensile strength better than their reference mix (RAC-CM) as displayed in Fig. 7.5. With respect to flexural strength, Fig. 7.6 shows that most RAC mixes produced comparable results compared to the control mixes (NAC-CM and RAC-CM). In contrast to compressive strength, SP has little effect on the tensile and flexural strength of RAC mixes.

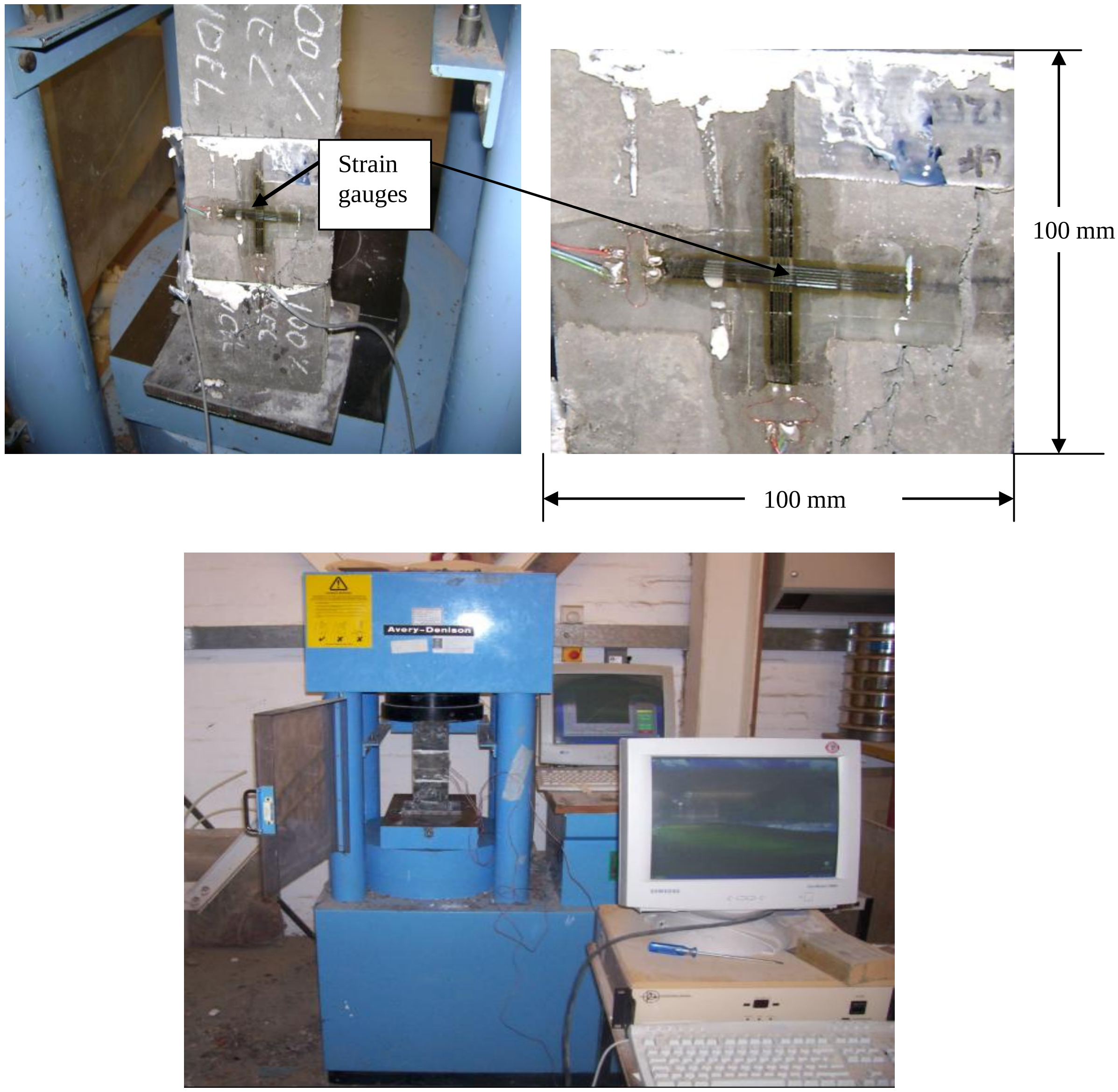
Tensile strength tended to increase slightly with increasing compressive strength. Data in Table 7.11 show the ratio of the tensile splitting strength to the compressive strength, it is in general marginally larger for NAC mixes than similar RAC concrete; it was between 13 to 19% for NAC and 14 to 16% for RAC concrete with PFA. Thus, the difference arising from different type of aggregate is not major. The ratio was also not shown to be crucially influenced by PFA replacement; however the results obtained were slightly high compared with that usually attained from normal strength concrete (without additives), which is about 10-12% and sometimes 15% (Neville 2003). The inspection of crushed samples, indicated the presence of sphere-like voids resulting from entrapped air bubbles of several sizes, in most cases (and irrespective of the aggregate type), no matter or whether not the samples were properly cast. This however, on one hand could be beneficial for practical aspects, such as the ability to resist stresses due to cyclic freezing and thawing in cold weather, but on the other could lead to decrease the ultimate strength and be harmful to its durability

(v) Modulus of elasticity of concrete when PFA replaces cement

The modulus of elasticity was measured as detailed in Chapter 5 Section 5.2 (e) and Tables 7.12 and 7.13 show a sample set of measurements. Figs 7.8 and 7.9 show samples of concrete tested for the modulus of elasticity.

Type: Wire 26ga -19 × 38 – 600V- 0.01PVC - 105C No 172619.
Resistance of bridge circuit = 120 Ω.
Gauge factor = 2.10.
Dummy gauge not needed as it is a thermal compensating gauge.
Load rate: 180 kN/min
Test temperature: 20 ± 2°C   Cube size: 100 × 100 × 100 mm
Jointing material: dental plaster

Fig. 7.8 A stack subjected to two load cycles controlled by a computer before crushing



Note:
A strain gauge used in the experiment was manufactured by Dearborn, USA; wired as quarter-bridge circuit into System 5000 data-logger; characteristics are:
Load rate: 180 kN/min
Test temperature: 20 ± 2°C
Cube size: 100 × 100 × 100 mm
Jointing material: Dental plaster

Fig. 7.9 Testing for the concrete modulus of elasticity: a sample under compression up to failure

The ratio of applied load to the loaded cross-sectional area of the specimen indicates the stress whereas the strain was obtained directly from the strain gauges. The results have been reproduced in graphical form; samples for NAC-CM and RAC-CM are shown in Figs 7.10 and 7.11.

Table 7.12 Modulus of elasticity measurements for NAC specimen (NAC-CM)

Vertical stress (N/mm^2)	Vertical strain	Horizontal strain
0	0	0
5.8	77	26
10	143	23
15	247	10
20	374	0
25	516	-22
30	667	-50
35	829	-76
40	1002	-111
45	1254	-262

Table 7.13 Modulus of elasticity measurements for RAC specimen (RAC-CM)

Vertical stress (N/mm^2)	Vertical strain	Horizontal strain
0	0	0
7.5	216	-47
10	273	-57
12.5	364	-78
15	449	-105
17.5	534	-135
20	619	-171
22.5	695	-228
25	767	-349
27.5	766	-626

From the data listed in Tables 7.12 and 7.13, stress-strain curves were drawn for Figs 7.10 and 7.11; from which the elastic moduli were obtained.

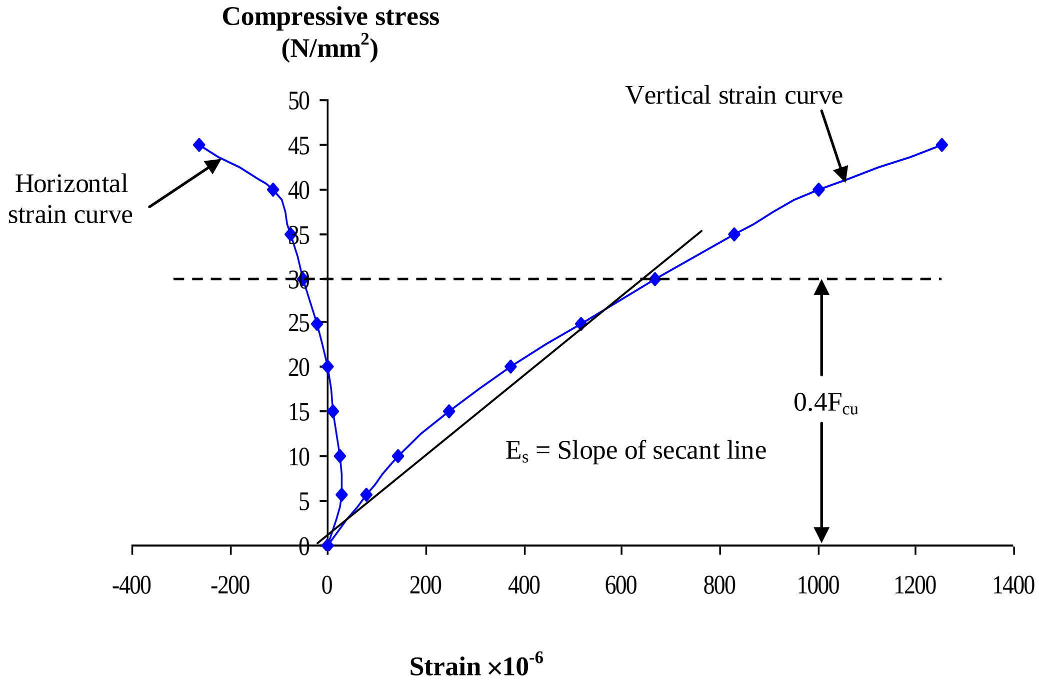


Fig. 7.10 Typical stress-strain curve for NAC specimen (NAC-CM)

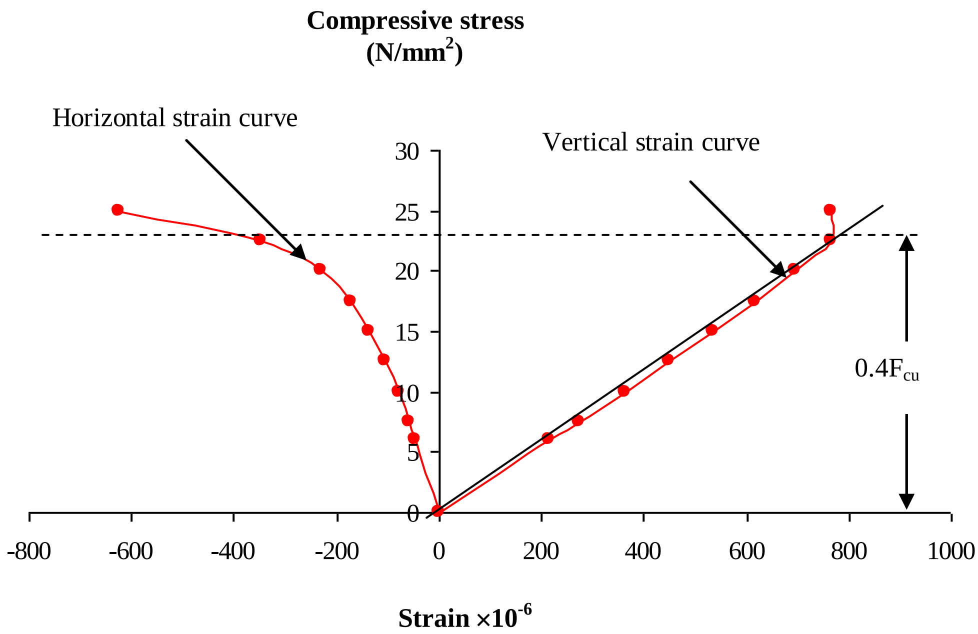


Fig. 7.11 Typical stress-strain curve for RAC specimen (RAC-CM)

The modulus of elasticity was taken as an average value from two samples, however it has been noticed that for both NAC and RAC mixes, this property is more variable than the compressive and tensile strength. RAC concrete stress-strain graphs are, similar to NAC concrete, typically v-shaped curves. Results of the elasticity tests for all mixes are given in Table 7.14.

Table 7.14 Modulus of elasticity and Poisson's ratio of mixes when PFA replacing cement

Mix	Code	PFA (%)	F_{cu} (N/mm^2)	E_s (GPa)	μ*	Reduction of E_s for RAC (%)	E_s (as % of NAC-CM)
1	NAC-CM	0	69	36.28	0.10	-	100
2	RAC-CM	0	52	28.30	0.28	23	78
5	NAC-16WR-0.8SP	0	74	51.40	0.39	-	142
6	RAC-16WR-0.8SP	0	64	40.00	0.30	22	110
9	NAC-30PFA	30	65	39.50	0.37	-	109
10	RAC-30PFA	30	50.8	24.93	0.50	37	69
11	NAC-16WR-0.8SP-30PFA	30	75.4	50.69	0.37	-	140
12	RAC-16WR-0.8SP-30PFA	30	61.6	37.16	0.40	27	102

*μ is Poisson's ratio

Results show that for all mixes the stress-strain relationships were found to be non-linear and that is in agreement with previous research (Neville 1995; Kumar & Prasad 2003; Jianzhuang *et al.* 2005; Khaldoun 2007). The elasticity modulus is known to increase generally over time, although at a different rate, often at a lower rate compared to compressive strength; for instance, E_s was enhanced by about 20% for 100% increase of strength using the same aggregate type and content (Zhou *et al.* 1995). Poisson's ratio of RAC concrete is slightly higher than NAC concrete, particularly when PFA is incorporated.

It is well known that the modulus of elasticity of concrete depends significantly on the modulus of elasticity of aggregates and the volumetric proportion of aggregate in the concrete (Neville 1995; Etxeberria *et al.* 2007). Recycled aggregates have a lower modulus of elasticity than natural aggregate (Frondistou 1977; Etxeberria *et al.* 2007). The type of coarse aggregate influences the modulus of elasticity of concrete; weaker aggregates tend to produce more ductile concrete than stronger aggregates (Beshr *et al.* 2003).

Test results obtained are in agreement with previous results (Aïtcin & Mehta 1990; Alexander 1995 & 1996; Ke-Ru *et al.* 2001; Ajdukiewicz & Kilszczewicz 2002; Beshr *et al.* 2003; Akash *et al.* 2007). Due to the large volume fraction it occupies in concrete, the inherent stiffness and the aggregate type exert the major influence on the elastic modulus of concrete (Ke-Ru 2001). Results of this study showed that the modulus of elasticity was influenced by the aggregate type (or more precisely by the aggregate strength) and the presence of PFA and SP in the mix. E_s generally increased as the compressive strength of concrete increased with both types of aggregates, but there was no agreement about the exact form of the relationship.

For superplasticized mixes, the measured E_s values were 40% and 2-10% more than the reference mix obtained for NAC and RAC concrete respectively. The increase of E_s however could be a result of the increased strength of the concrete matrix and the strength of the interfacial zone which in turn increased the bond between the aggregate particles and the matrix. The strength of the aggregate itself may also have an effect on the ultimate value of E_s and that is the likely reason behind the lower E_s of all RAC concrete mixes relative to NAC ones.

For non-superplasticized mixes, the 30% PFA cement replacement increased E_s for NAC by just 9% (109% - 100%), while it was reduced by 9% (78% - 69%) for similar RAC concrete; the reduction of E_s can be attributed to the reduced amount of cement; thus simply indicating that use of SP when PFA is used is significant for good quality RAC.

It is evident that E_s for the superplasticized RAC concrete was better than E_s for the NAC reference mix; E_s was greater by 2% for non-superplasticized RAC and by 10% for superplasticized RAC, but with respect to aggregate type, all RAC mixes express reductions of E_s when compared with similar NAC concrete. The reduction was between 22 and 37%. This situation was predicted as recycled aggregates are more prone to deformation than natural aggregate.

When mixes with and without PFA are compared, PFA is likely to have contributed to the reduction of E_s. From the data listed the fraction of reduction contributed by the inclusion of PFA was about 14% (37% - 23%) and 5% (27% - 22%) for non-superplasticized and superplasticized mixes respectively. However, this is in agreement with the observation made by several previous studies (see Chapter 3 for more details). The lowest E_s value obtained for RAC incorporating PFA was about 25 GPa; this value is however similar to the performance of lightweight concrete and higher than the findings of earlier results from ordinary concrete incorporating natural and man-made aggregates. Moreover, modulus might be further increased over time. Another observed feature is that both NAC and RAC mixes with and without PFA have exhibited nearly linear elastic behaviour.

(vi) Modulus of elasticity of concrete when PFA replaces cement estimated by other empirical methods

The modulus of elasticity of concrete is usually related to its compressive strength or density. The following expressions are recommended by BS 8110 (1985) and the ACI 318/318R (1992) as well as the Australian Standard (AS 3600-156-1994) to estimate the modulus of elasticity of NAC concrete as a short-cut to avoid direct expensive and time-consuming measurements. The aim of the analysis in this part is to check the suitability of these models for RAC made with PFA and SP.

$E_s = 1.7 \times 10^{-6} \rho^2 (F_{cu})^{0.33}$ (in GPa) suggested by BS 8110 and, (Eqn 7.1)

$E_s = 0.043 \times \rho^{1.5} (F_c)^{0.5}$ (in GPa) suggested by ACI 318/318R (1992) or, (Eqn 7.2)

$E_s = 0.0384 \rho^{1.5} (F_{cu})^{0.5}$ (in GPa) suggested by AS 3600-156-194 (Eqn 7.3)

Where E_s, ρ, F_c and F_{cu} are modulus of elasticity (GPa), the concrete density (kg/m^3), cylinder compressive strength (N/mm^2) and cube compressive strength (N/mm^2) respectively. Table 7.15 lists measured and estimated E_s values for NAC concrete.

Table 7.15 Measured and estimated E_s values for NAC concrete samples

Mix	Code	28 day F_{cu} (N/mm^2)	Density (kg/m^3)	Measured E_s (GPa)	E_s (GPa) by:	
					BS8110	ACI318
1	NAC-CM	69.0	2393	36.28	39.4	37.3
5	NAC-16WR-0.8SP	74.0	2316	51.40	37.7	36.8
9	NAC-30PFA	65.0	2240	39.50	33.8	32.8
11	NAC-16WR-0.8SP-30PFA	75.4	2242	50.69	35.6	35.4

It seems that BS 8110 and ACI 318/318R models produce comparable estimates of E_s for all NAC concrete mixes. When both are compared with measured E_s for the reference mix the result is also comparable, in particular for ACI 318. However, the two standards conversely underestimate for mixes made with SP, PFA or with the combination of them, therefore it could be concluded that both standards are unsuitable for estimating E_s for NAC concrete made with chemical and mineral admixtures.

For the RAC concrete, the following expressions can be used:

$E_s = 1.9 \times 10^5 \text{ x } (\rho / 2300)^{1.5} \text{ x } (F_{cu}/2000)^{0.5}$ (in N/mm^2) (Eqn 7.4)

Equation 7.1 was suggested by (Kakizaki *et al.* 1988), and;

$E_s = 9100 \times (F_{cu}+ 8)^{1/3} \times (F_{cu}/2400)^2$ (in N/mm^2) (Eqn 7.5)

Equation 7.2 was suggested by (Zilch & Roos 2001), where E_s, ρ, and F_{cu} are modulus of elasticity (GPa), concrete density (kg/m^3), and cube compressive strength (N/mm^2) respectively. Table 7.16 lists measured and estimated E_s values for NAC concrete.

Table 7.16 Measured and estimated E_s values for RAC concrete samples

Mix	Code	28 day F_{cu} (N/mm^2)	Density (kg/m^3)	Measured E_s (GPa)	E_s (GPa) by:	
					Kakizaki *et al.*	Zilch & Roos
2	RAC-CM	52.0	2280	28.30	30.24	32.15
6	RAC-16WR-0.8SP	64.0	2343	40.00	34.95	36.10
10	RAC-30PFA	50.8	2333	24.93	30.93	33.44
12	RAC-16WR-0.8SP-30PFA	61.6	2204	37.16	31.28	31.56

Similar to NAC, the two expressions used to estimate E_s have produced similar results for all RAC concrete mixes. For the reference mix, the expressions resulted in E_s values as good as the measured result, but the two E_s expressions (Eqns 7.4 & 7.5) overestimate the E_s of the RAC with PFA yet underestimate E_s for RAC with SP and that of RAC with SP and PFA, although they were originally suggested for RAC. Therefore it could be concluded that both expressions are inappropriate for estimating E_s of RAC concrete made with chemical and mineral admixtures. This may be because these expressions were derived from data here which did not include mixes produced with chemical or mineral admixtures.

(vii) Density and pulse velocity of concrete when PFA replaces cement

The concrete density of samples and the velocity of longitudinal compression wave pulses through concrete were measured just before crushing of cubes at 28, 56, and 90 days. Table 7.17 presents the results.

Table 7.17 Density and pulse velocity of concrete made without and with PFA replacing cement

Mix	Code	PFA (%)	Density (kg/m^3) and pulse velocity (km/s) after: 28 days	56 days	90 days
1	NAC-CM	0	2393 (4.85)	2288 (4.76)	2278 (4.86)
2	RAC-CM	0	2280 (4.45)	2305 (4.51)	2292 (4.50)
5	NAC-16WR-0.8SP	0	2316 (4.85)	2314 (5.04)	2322 (5.1)
6	RAC-16WR-0.8SP	0	2343 (4.54)	2365 (4.88)	2338 (4.69)
9	NAC-30PFA	30	2240 (4.72)	2203 (4.76)	2185 (4.88)
10	RAC-30PFA	30	2233 (4.52)	2171 (4.70)	2182 (4.58)
11	NAC-16WR-0.8SP-30PFA	30	2242 (4.9)	2181 (5.10)	2182 (4.93)
12	RAC-16WR-0.8SP-30PFA	30	2204 (4.55)	2200 (4.60)	2204 (4.64)

The density of hardened concrete has shown to be affected by the incorporation of PFA; Figs 7.12 and 7.13 demonstrate the reduction in concrete density due to 30% PFA cement replacement.

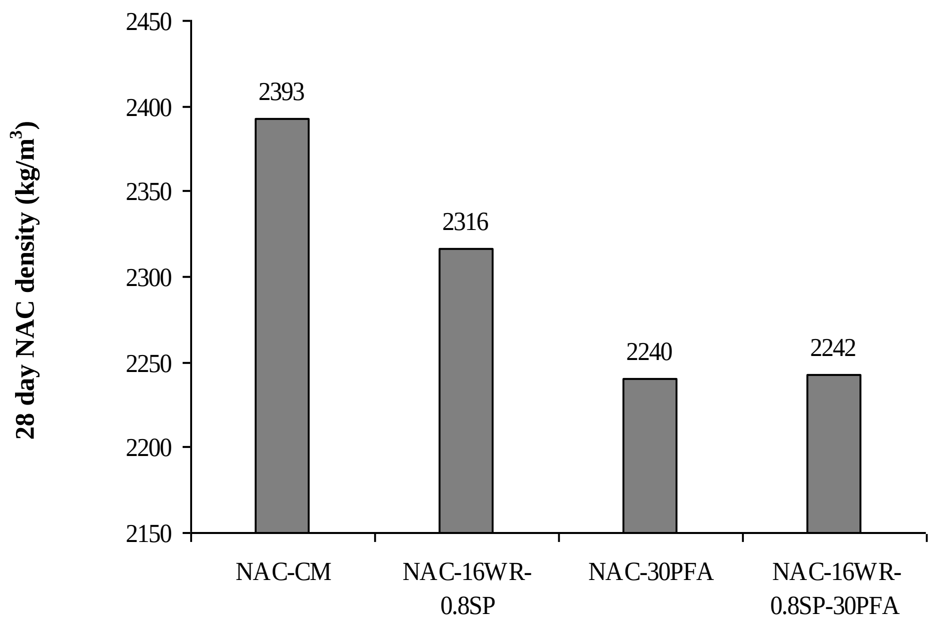


Fig. 7.12 Influence of PFA cement replacement on the bulk density of NAC concrete

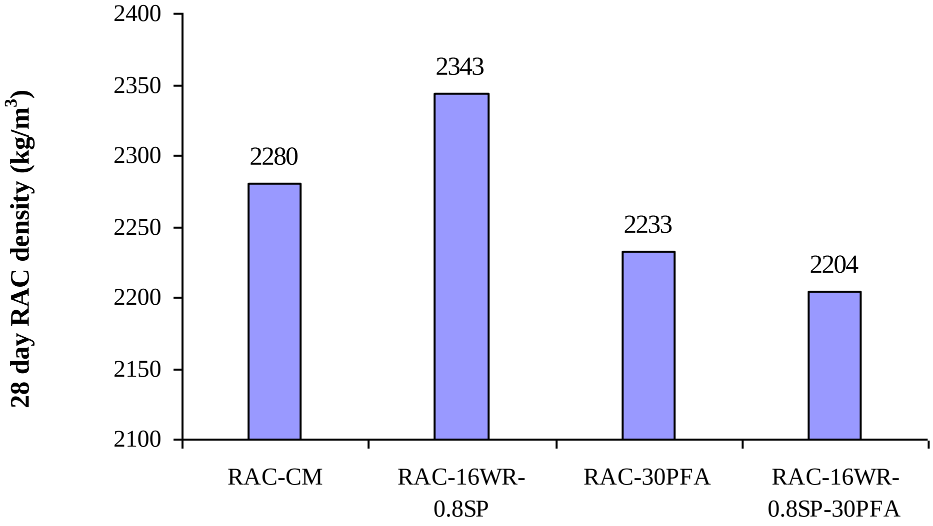


Fig. 7.13 Influence of PFA cement replacement on the bulk density of RAC concrete

Table 7.22 shows a comparison between concrete mixes made without and with PFA arranged in a way to show the effect of substituting 30% of the cement content for PFA on density, compressive strength, and modulus of elasticity of concrete after 28 days' curing.

Table 7.18 Influence of PFA on the density and compressive strength of concrete

Mix without PFA	Mix with PFA	Reduction of 28 day density (%)	Reduction of 28 day F_{cu} (%)	Reduction of 28 day E_s (%)
NAC-CM	NAC-30PFA	6.5	6	- *
RAC-CM	RAC-30PFA	2	2.5	12
NAC-16WR-0.8SP	NAC-16WR-0.8SP-30PFA	3	- *	1.5
RAC-16WR-0.8SP	RAC-16WR-0.8SP-30PFA	6	4	7

*E_s was not reduced but increased instead

Results in Table 7.18 showed that the replacement of 30% cement content by PFA decreases the compressive strength insignificantly; the reduction ranged from 0% to less than 6% while the reduction of elastic modulus ranged from 0% to 12%. However, reductions were generally marginal and counterbalanced by the measured reduction of concrete density (2-6%), which implies lighter sections. This merit can be added to several other advantages of incorporating PFA in concrete as detailed in Chapter 3.

(viii) Relationship between density and compressive strength

The data obtained were re-arranged in Table 7.19 and shwnin graphical form in Fig. 7.14.

Table 7.19 Compressive strength and density of mixes

Mix	F_{cu} (N/mm^2)	Density (kg/m^3)
RAC-16WR-0.8SP	64	2343
	72	2365
	73	2338
RAC-30PFA	50.8	2233
	52.5	2171
	59.5	2182
RAC-16WR-0.8SP-30PFA	61.6	2204
	67.5	2200
	69	2205

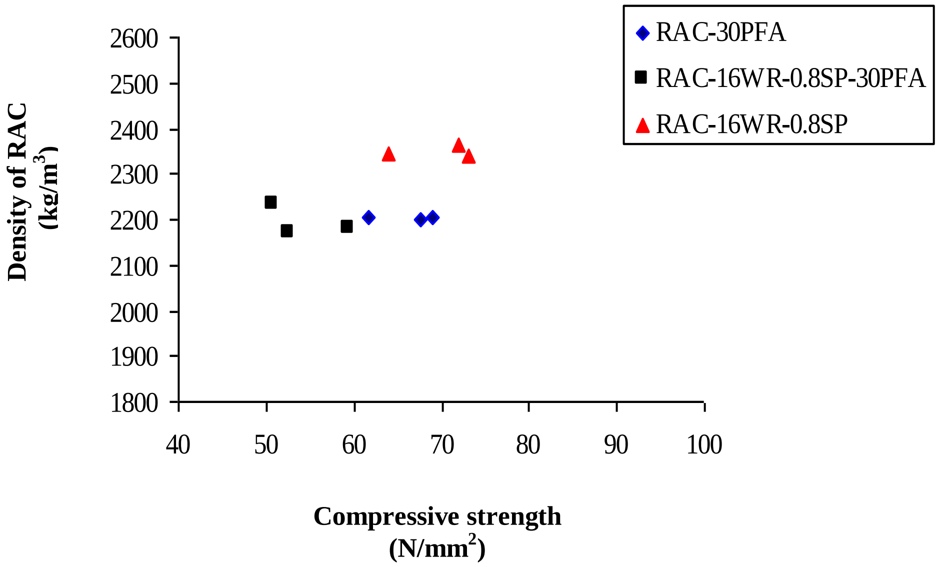


Fig. 7.14 Density of RAC mixes with admixtures at different ages against the corresponding compressive strength.

The relationship between the density of RAC concrete (ρ) with admixtures (SP or PFA or both) and the compressive strength is not well defined, the measured density values are widely scattered. The trend-line linear relationship based on the data in Fig. 7.14 shows that the density is given by: $\rho = 5.6\ F_{cu} + 1895$ (kg/m^3), where F_{cu} is the compressive strength of RAC (N/mm^2), however the correlation coefficient $R^2 = 0.334$ indicates very low correlation. This implies it is unreliable to estimate the compressive strength from the density of this type of concrete.

According to the relationship (Xiao *et al*. 2006) which was developed as a result of studying data collected from a large number of experimental research projects on RAC, published between 1985 and 2004, the compressive strength of RAC can be estimated from the knowledge of its density, by the expression:

$F_{cu} = 0.069\ \rho - 116.1$ (Eqn 7.6)

Where, F_{cu} is the compressive strength (N/mm^2), and ρ is the density (kg/m^3), with a correlation coefficient $R^2 = 0.92$. The estimated compressive strength of RAC concrete is given in Table 7.20.

Table 7.20 Measured and estimated F_{cu} values for RAC concrete samples

Mix	Code	PFA (%)	Measured 28 day F_{cu} (N/mm^2)	Density (kg/m^3)	Estimated F_{cu} (N/mm^2) by Xiao *et al.*
2	RAC-CM	0	52	2280	41.22
6	RAC-16WR-0.8SP	0	64	2343	45.56
10	RAC-30PFA	30	50.8	2333	44.88
12	RAC-16WR-0.8SP-30PFA	30	61.6	2204	35.98

It is obvious that the expression suggested (Xiao *et al*. 2006) underestimates the real compressive strength of RAC concrete produced with SP and PFA. This again supports the conclusion that it is unreliable to estimate the compressive strength from the density of this type of concrete. The difference maybe because the raw data used to develop the expression were derived from non-superplasticized mixes in which only cement was used as a binder.

A comparison between the measured mechanical properties of PFA concrete and the red granite dust (RGD) concrete as well as those obtained by some empirical numerical expressions of international codes and researchers has been prepared at the end of the following section.

7.3 RAC CONCRETE PRODUCED WITH RGD PARTIALLY REPLACING CEMENT

(a) Introduction

A major factor affecting the construction industry is the cost of building materials. Therefore there is a growing realization, due to increasing cost, of the importance of research into utilising locally available wastes and by-products as alternatives to natural materials. On this background, literature shows several naturally occurring powders such as limestone and chalk and industrial by-products as well as different ashes produced from burning natural agricultural products were employed in concrete mixtures either to supplement or substitute cement. Different reasons were cited; most were technical and economical.

The addition of a certain material at a particular level to supplement or substitute cement could have positive or alternatively negative implications on various properties of the end product. It has been shown that for most of the cases in which these materials were used, the improvement mechanism was either chemical by which the chemical reaction with cement contributes to improvement, or physical in which the filling effect of fine grained particles had considerable influence on the quality of concrete. However both mechanisms are also possible in combination.

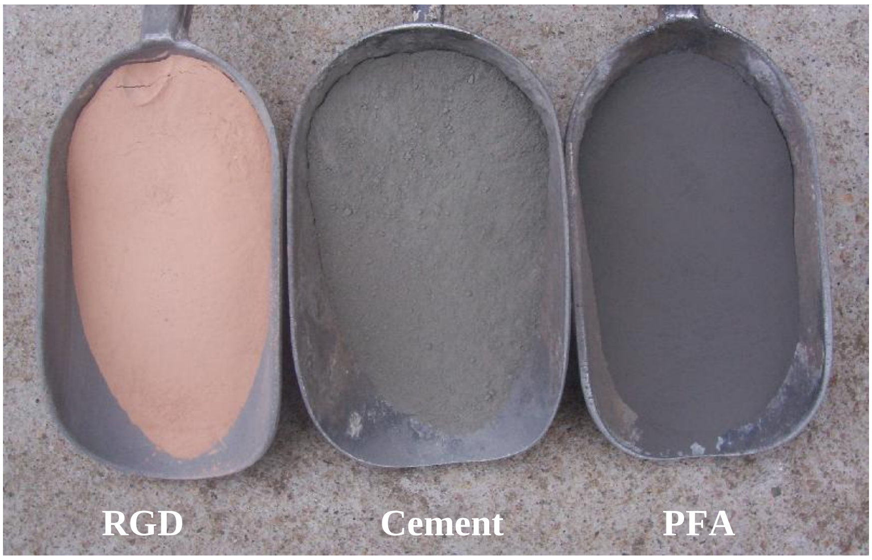


Fig. 7.15 RGD, cement and fly ash samples

Cloburn quarry near Lanark, Scotland was selected as a local source of the RGD by-product of the granite crushing process (see Fig. 4.1). The process includes crushing of a large granite mass to produce smaller size aggregate particles; however the cutting of larger rock and the crushing process results in smaller size grains and powder (RGD) which need to be separated by vacuum filter system to ensure the desired quality of aggregate. As a consequence, the collected powder, which can be as much as 1-2% by mass of the coarse aggregate production, accumulates over time and becomes difficult to accommodate in the quarry due to limited space, and health concerns inside the quarry site and local community, particularly on windy days in which this fine dust spreads to the surrounding area unless the RGD piles are sprayed with large amounts of water.

On the other hand if these by-products are going to be dumped in landfill sites, it will cost a considerable amount of money for transport and disposal charges. It is understandable that this waste material is causing a problem to the aggregate producer and the environment. Therefore, finding alternative outlets to contain or absorb this material is indeed a major concern. In this part of the study, the potential for recycling RGD in concrete was investigated. If the employment of this by-product is proven to be beneficial for concrete mixes just like other frequently used mineral admixtures, then large-scale use can address the aforementioned problems, reduce demand on sources of primary materials, and provide cost savings.

BS 882-2-1983 defines crushed rock dust, or simply dust, as aggregate particles passing a 75 µm aperture sieve. In addition to visual inspection, the tested RGD sample, by SEM images (Chapter 2), shows that the material grain size was far below this limit; therefore it can be classed as a very fine powder.

There is little published literature on the use of granite dust in concrete. The aim of this part of the thesis is to investigate the potential of exploiting this, locally abundant, by-product in new concrete; in other words, to check the possibility of beneficially recycling RGD in concrete products. The experimental investigation will be focused on the basic properties of concrete in which RGD partially replaces cement. Comparison of PFA and RGD concretes will also be made. The estimation of mechanical properties of concrete with the available models and standards will be studied.

(b) Selection of the best RGD cement replacement level

In contrast to PFA mixes, the ideal level of replacement for concrete mixes with RGD is not known. In the case of PFA mixes, concrete was produced with an optimum PFA content of 30% of the cement on the basis of careful inspection of the published data. Therefore, the first rational step is to determine the best possible level of RGD before proceeding to further testing. In this context, the most suitable level will be selected based on workability and the strength of concrete attained by trial mixes.

It has been noticed that RGD is a material similar to PFA (see Chapter 4 for more details), therefore it is sensible to assume similar behaviour, and thus the same mix proportion used for PFA mixes can be adopted for RGD. Mix proportions of concrete contains RGD at different levels are given in Table 7.21; full mix designs are available in Appendix 2.

Table 7.21 Standard mixes with RGD at different levels

RGD (%)	Mass (kg/m^3)				
	Cement	RGD	Water	Coarse aggregate	Fine aggregate
0	425	0	170	1280	545
20	372	93	160	1270	545
30	345	145	155	1265	540
40	312	208	150	1250	535
50	280	280	145	1230	525

For selection purposes, a total of five NAC concrete trials, with RGD partially replacing cement, were produced with 0.8% SP and 16% WR (as it was previously proved that these are the best combinations for the material used in this study). The only variable in these mixes was the percentage of RGD; percentages of 0, 20, 30, 40 and 50% RGD were adopted. All other components were kept constant. These mixes were coded as NAC- X RGD- 0.8SP with X representing the percentage of RGD (the ratio of RGD mass to cement mass).

(i) Workability of mixes used for selection of optimal RGD cement replacement level

Table 7.22 displays mixes used for the selection of the best RGD cement replacement level and their workability.

Table 7.22 Workability of mixes used to determine the best RGD percentage

Mix	RGD (%)	WR (%)	SP (%)	w/c	Slump (mm)	Vebe time (s)	Visual observations
NAC-CM	0	0	0	0.40	15	12	Uniform mix
NAC-0RGD-0.8SP	0	16	0.8	0.34	20	10	Uniform mix
NAC-20RGD-0.8SP	20	16	0.8	0.34	25	9	Uniform, cohesive mix
NAC-30RGD-0.8SP	30	16	0.8	0.34	20	11	Uniform, cohesive mix
NAC-40RGD-0.8SP	40	16	0.8	0.34	25	8	Uniform, less cohesive mix
NAC-50RGD-0.8SP	50	16	0.8	0.34	30	6	Lack of cohesion, signs of segregation

The relationship between workability and the RGD level is shown in Fig. 7.16.

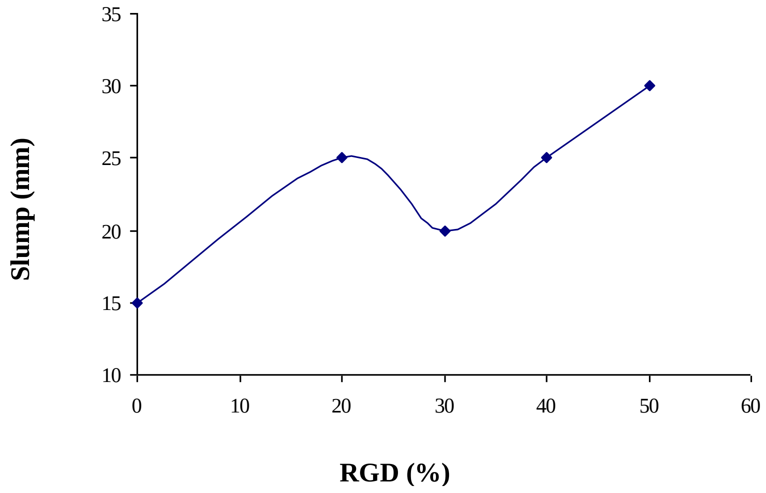


Fig. 7.16 Effect of RGD content on workability of NAC mixes

Fig. 7.16 shows the variation in slump *versus* RGD content for NAC mixes listed in Table 7.22. The graph indicates that as the RGD content increases, the slump initially increased; this could be attributed to the SP's dispersion influence, however beyond 20% cement replacement, slump decreased to reach its minimum at 30% RGD cement replacement but was not less than that of the reference mix. Slump and workability in general express the ease of concrete handling and placing; this can be much affected by the available voids and the movement of grains over each other and, of course, by the shape and texture of particles amongst other factors.

The physical reason for slump reduction within the range of 20 to 30% RGD is possibly due to rearrangement of the grains; it seems that at this level the packing density of wet concrete achieved its highest value. Beyond this level the maximum packed structure is collapsed again and the slump starts to increase within the range below 50% RGD replacement. The increased slump with the increase of RGD level is likely to be due to the increased free water content in the voids due to the lower absorption of RGD grains when compared with cement; cement reacts chemically with water and when there is more cement (or less RGD) there will be less free water for grain lubrication. In contrast no reaction of RGD with water takes place; water is absorbed by grains and their fluidity is increased when there is less cement in the mix. However, further increase of RGD content is predicted to result in lack of cohesion due to the reduction of cement content. However, evidently, below 50% substitution, results show that it was possible to maintain adequate concrete workability, with the help of SP.

(ii) Mechanical properties of mixes used for selection of optimal RGD cement replacement level

Results of compressive, tensile and flexural strengths are given in Table 7.23.

Table 7.23 Strengths of mixes used to determine the best RGD percentage

Mix	Compressive strength after: (N/mm^2)			Tensile splitting strength after: (N/mm^2)			Flexural strength after: (N/mm^2)	
	28 (d)	56 (d)	90 (d)	28 (d)	56 (d)	90 (d)	28 (d)	56 (d)
NAC-CM	69.0	71.2	76.0	4.70	3.78	4.60	8.10	9.75
NAC-0RGD-0.8SP	74.0	89.7	91.7	5.78	7.17	7.05	11.3	11.3
NAC-20RGD-0.8SP	70.7	71.0	72.0	2.97	3.79	3.64	7.50	7.46
NAC-30RGD-0.8SP	72.0	76.3	73.8	4.59	4.50	5.40	7.56	6.73
NAC-40RGD-0.8SP	54.5	63.9	65.5	3.50	3.96	3.85	7.78	6.48
NAC-50RGD-0.8SP	45.8	50.0	46.7	2.45	3.04	3.51	6.20	6.60

The compressive strength data at 28 days in Table 7.23 are shown in Fig. 7.18. The superplasticized NAC concrete with up to 30% RGD replacing cement achieved a compressive strength of 72 N/mm^2, more than the target mean strength (TMS) of the control mix (63 N/mm^2). The test data at later ages exhibited the same trend as that obtained at 28 days.

Concrete cubes and fracture surfaces with different RGD content are shown in Fig. 7.17. Shearing of aggregate grains in less RGD content concrete indicating higher strength of the matrix as can be seen in NAC-20RGD-0.8SP and NAC-30RGD-0.8SP, higher RGD (40 and 50%) content exhibited bond failure and less strength.

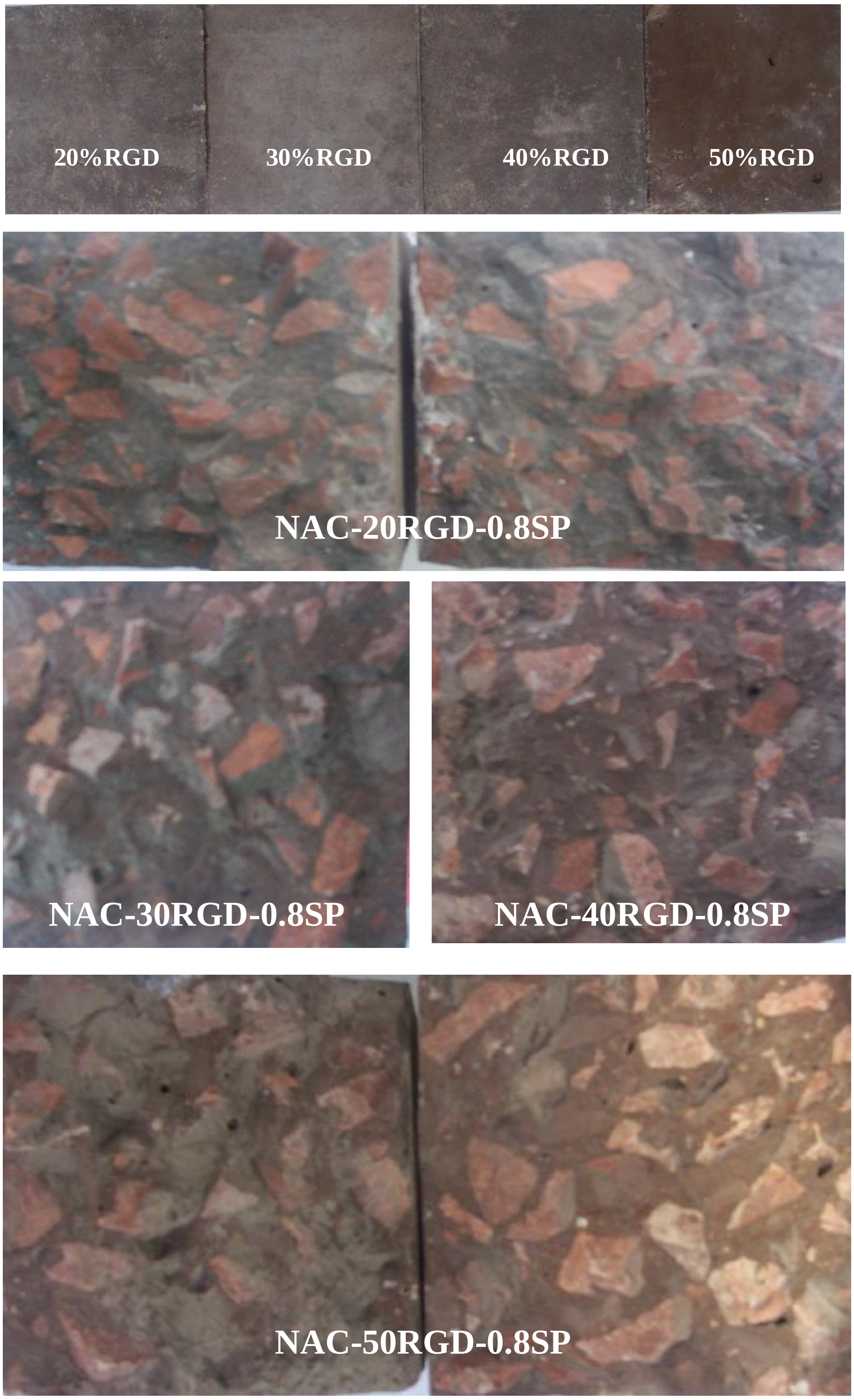


Fig. 7.17 Concrete cubes and fracture surfaces with different RGD content

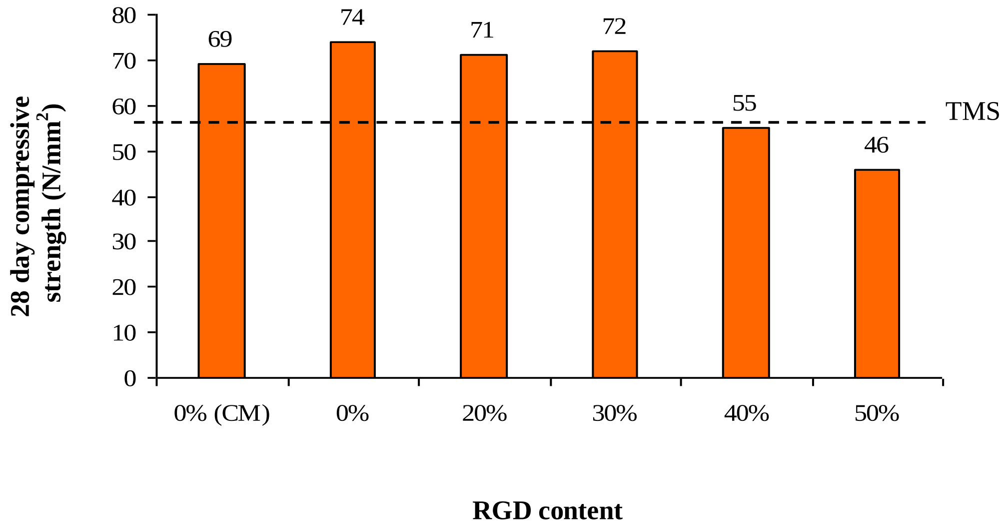


Fig. 7.18 Compressive strength of different RGD concretes at 28 days

However, the superplasticized concrete without RGD was better than that with RGD; but the concrete mixes with 20 and 30% RGD were better than the reference mix NAC-CM (the mix without SP and RGD). Among all the mixtures tested, the mix with 30% RGD has attained the highest 28 day compressive and tensile strength, and a flexural strength accounting for 93% of the reference mix value.

With respect to workability, results showed that the mix with 30% RGD was uniform and the measured slump (20 mm), although the lowest, nevertheless represents an average value of the design slump (10-30 mm); the Vebe time of 11 s was within the design limit (6-12 s). The variation of compressive and tensile strength of NAC mixes produced at different levels of RGD replacement is shown in Figs. 7.18 and 7.19.

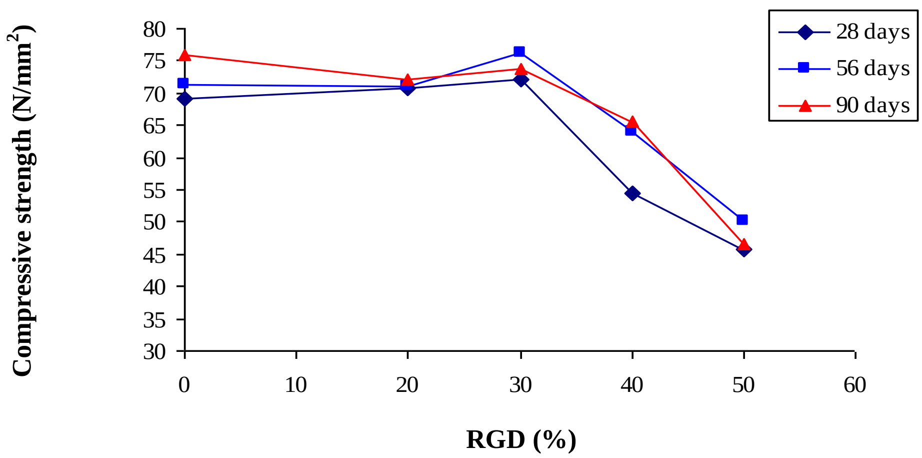


Fig. 7.19 Compressive strength of different RGD concretes at different ages

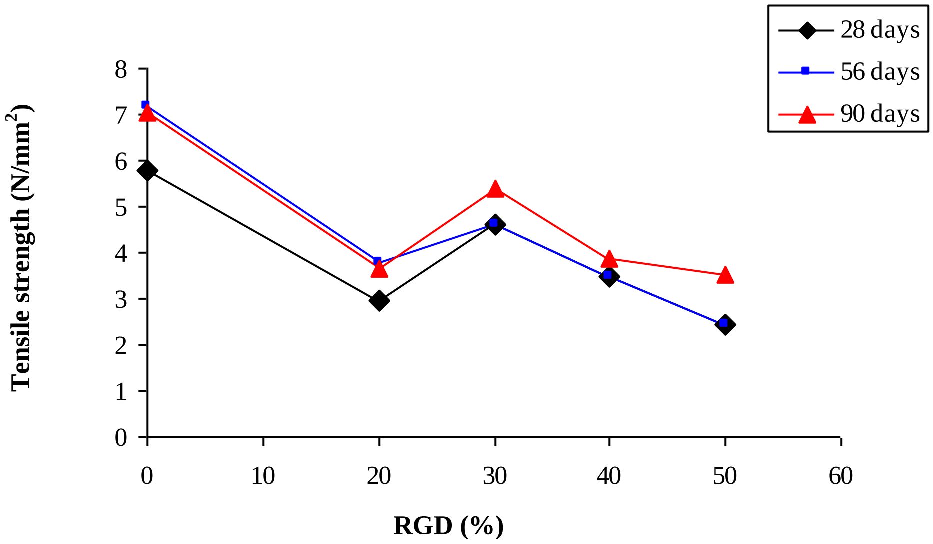


Fig. 7.20 Tensile strength of different RGD concretes at different ages

Fig. 7.19 clearly indicates the highest compressive strength was attained by the 30% RGD mixture at all ages, beyond which the strength of concrete generally decreased with an increase in RGD content due to high cement replacement; concrete strengths with 40 and 50% RGD were substantially lowered when compared to the reference mixture. A similar trend, as displayed in Fig. 7.20, has been noticed for the tensile splitting strength as well as flexural strength. In addition to the aforementioned reason, *i.e.* the decreased amount of cement, the decrease in tensile strength may also be attributed to the increased amount of free water due to the lower propensity for absorption of RGD; this was despite the fact that a constant w/c ratio was used for each mix. As a result the higher RGD content mixes became wet and lean which caused low strength values after hardening.

On the basis of the achieved strengths, the 30% RGD cement replacement level can be selected for further testing of concrete involving RGD. This supports the assumption made earlier that RGD is similar to PFA; 30% PFA was used in the first part of this chapter. This however, provided the motivation for further testing using the same mix design used earlier for the PFA mixes.

(iii) Density and pulse velocity of mixes used for selection of optimal RGD cement replacement level

The density and pulse velocity of the tested samples are displayed in Table 7.24.

Table 7.24 Density and pulse velocity of mixes used to determine the best RGD percentage

Mix	Density (kg/m^3) and pulse velocity (km/s) after:		
	28 days	56 days	90 days
NAC-CM	2393 (4.85)	2288 (4.76)	2278 (4.86)
NAC-0RGD-0.8SP	2316 (4.85)	2314 (5.04)	2322 (5.10)
NAC-20RGD-0.8SP	2216 (4.88)	2203 (4.92)	2207 (4.91)
NAC-30RGD-0.8SP	2193 (5.00)	2237 (4.94)	2223 (4.97)
NAC-40RGD-0.8SP	2189 (4.73)	2185 (4.90)	2190 (4.96)
NAC-50RGD-0.8SP	2176 (4.65)	2184 (4.82)	2181 (4.87)

Values in brackets are pulse velocities

The variation of bulk density of concrete with the level of RGD content in the binder (cement + RGD) is shown graphically in Fig 7.19.

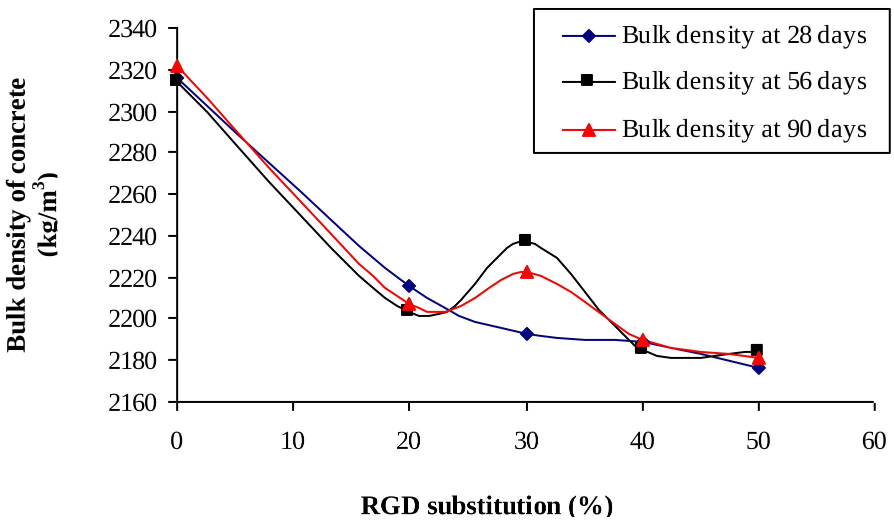


Fig. 7.21 Variation of concrete bulk density with binder RGD content

The data show that both density and pulse velocity were, in general, decreasing with the addition of RGD, however, the concrete density at 30% RGD content is noticeably improved 'locally' at 56 and 90 days, but not at 28 days. This is believed to be a result of the improved pore structure, at later ages with 30% addition level, resulting from the continued hydration reaction of cement and the refining or filling effect of RGD. In other words, the increased content of calcium silicate hydrate (CSH) due to the permanent hydration and the bulk filling nature of RGD particles which end up filling the voids within the concrete increasing its volume thus reducing its density beyond 56 days. Despite this local improvement of density at 30% RGD addition at later ages, the density is generally still below that of the standard mix by 2.2 and 2.4% at 56 and 90 days respectively. The local increase of density is in harmony with the increase of strengths; the density was increased beyond 20% replacement to reach its local peak at 30% replacement. Extensive decline of density was again observed beyond about 30%. The pulse velocity followed an almost identical trend, the maximum was observed at 30% RGD. Fig. 7.22 shows the variation of pulse velocity throughout concrete with varying level of RGD cement substitution.

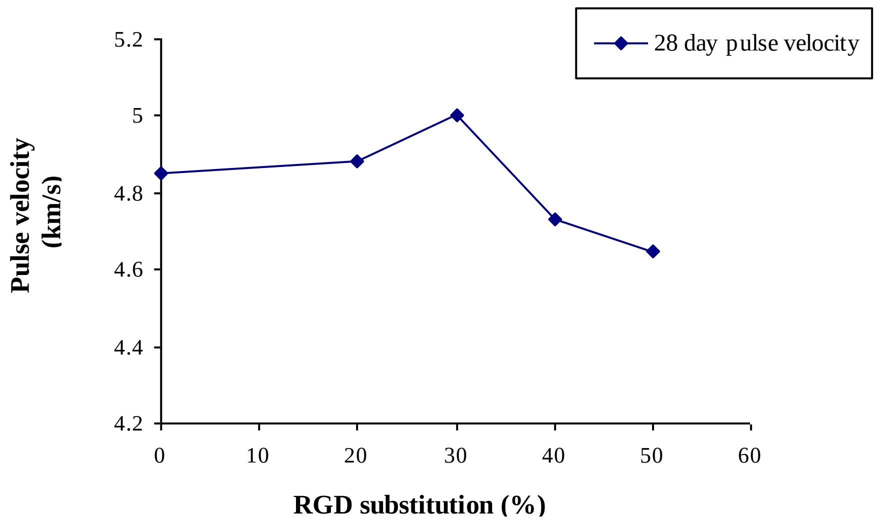


Fig. 7.22 Variation of pulse velocity of concrete with the level of RGD cement substitution

The ultrasonic pulse velocity test makes use of the relationship between wave velocity in concrete and stiffness. The wave propagation velocity or pulse velocity (v_b) is related to concrete stiffness (E_d) and mass density (ρ) by the relation in Eqn 7.7.

$$(v_b)^2 = E_d/\rho \qquad \text{(Eqn 7.7)}$$

However, E_d in this case is the dynamic modulus of elasticity which is often higher than the static modulus, although a few studies have shown the opposite (Ala 2004).

The E_d can be calculated if the concrete Poisson's ratio (μ) is known (or estimated) by the relationship in Eqn. 7.8.

$$E_d = (1 - \mu)\, \rho\, (v_b)^2 \qquad \text{(Eqn 7.8)}$$

The time for a pulse to travel a known distance (the distance between transmitter and receiver of pulse) is determined and the quality of concrete can be derived; in this experiment the frequency of the transmitter used was 54 kHz. The test is covered by BS 1881: Parts 202 and 203.

Table 7.25 Classification of the quality of concrete on the basis of pulse velocity

Pulse velocity (km/s)	Quality of concrete
> 4.5	Excellent
3.5-4.5	Good
3.0-3.5	Doubtful
2.0-3.0	Poor
< 2.0	Very poor

Considering the measured pulse velocity, the range was from 4.65 to 5.00 km/s; therefore, all RGD mixtures can be rated excellent according to the criteria given in Table 7.25. Again, this may be due to overall improvement of pore structure.

As the best RGD level was selected, examination of the performance of RAC using this level was possible. NAC concrete mixes were also produced as control mixes.

(c) Concrete mixes with RGD replacing cement

(i) General

To investigate the possibility of utilising RGD in NAC and RAC concrete, four other concrete mixes in which 30% of their cement was replaced by RGD were produced. Two of them (Mixes 13 and 14) were standard mixes designed to achieve 50 N/mm^2 at 28 days with a slump of 10-30 mm and Vebe time of 6-12 s for NAC and RAC mixes, in accordance with the Building Research Establishment mix design method (BRE 1992) that was especially prepared for fly ash concrete mixes, at a level of 30% RGD to partially replace cement particularly prepared for fly ash concrete mixes on the basis of the assumption that RGD is similar to PFA. In the other two mixes (Mixes 15 and 16), the water content of the above standard mixes was reduced by 16% combined with 0.8% SP (as proven optimal for the materials at hand). Mix design data sheets are available in Appendix 2. All RGD mixes were kept in similar proportions to PFA mixes so that legitimate comparisons could be made; keeping in mind that for the superplasticized mixes, only the mixing option with 0.8% SP and 16% WR was used as it was proven to be the best combination, as listed in Table 7.26.

Table 7.26 Concrete mixes without and with 30% RGD replacing cement

Mix	Code	SP (%)	WR (%)	RGD (%)
Mixes without RGD				
1	NAC-CM	0	0	0
2	RAC-CM	0	0	0
5	NAC-16WR-0.8SP	0.8	16	0
6	RAC-16WR-0.8SP	0.8	16	0
Mixes with 30% RGD				
13	NAC-30RGD	0	0	30
14	RAC-30RGD	0	0	30
15	NAC-16WR-0.8SP-30RGD	0.8	16	30
16	RAC-16WR-0.8SP-30RGD	0.8	16	30

Again basic mechanical properties of concrete were evaluated for RGD concrete specimens under study in accordance with the relevant British and European standards. Tests include compressive strength, tensile splitting strength, flexural strength (modulus of rupture) and modulus of elasticity. The results of these properties were analysed in comparison to similar PFA concrete specimens.

(ii) Workability of concrete mixes without and with 30% RGD replacing cement

Table 7.30 shows the workability measurements (standard slump and Vebe time) and the visual observations made on the produced mixes.

Table 7.27 Workability of mixes with 30% RGD replacing cement

Mix	Code	w/c	w/b	SP (%)	WR (%)	Slump (mm)	Vebe (s)	Visual observations
Mixes without RGD								
1	NAC-CM	0.40	0.4	0	0	15	12	Uniform mix
2	RAC-CM	0.40	0.4	0	0	10	14	Uniform mix
5	NAC-16WR-0.8SP	0.34	0.34	0.8	16	15	10	Uniform, cohesive mix
6	RAC-16WR-0.8SP	0.34	0.34	0.8	16	25	7	Uniform, cohesive mix
Mixes with 30% RGD								
13	NAC-30RGD	0.4	0.32	0	0	20	9	Uniform, needs 2 min compaction time
14	RAC-30RGD	0.4	0.32	0	0	15	13	Uniform needs 3 min compaction time
15	NAC-16WR-0.8SP-30RGD	0.34	0.27	0.8	16	55	5	Cohesive and uniform
16	RAC-16WR-0.8SP-30RGD	0.34	0.27	0.8	16	45	7	Cohesive and uniform

Data presented in Table 7.27 show that it was possible to produce concrete made of 30% RGD cement replacement with acceptable workability; little extra compaction was needed for no-SP mixes. Comparing the slump of mixes with 30% RGD and the control mixes, a marginal improvement was observed; that however could be interpreted as indicative of a reduction of absorption capacity of the newly formed binary binder *i.e.* 70% cement and 30% RGD; this implies that RGD is less absorbent than cement. The workability was much improved in superplasticized mixes despite the water reduction. However, when compared with similar PFA concrete, RGD mixes exhibited lower slump; that may be linked to the shape of the two materials; PFA grains were spherical while RGD grains were rough, irregular and angular, as appeared from SEM images (see Chapter 4).

Another point maybe worthy of mention is that the appearance of the outer shell of RGD concrete is not similar to PFA concrete; the discoloration of PFA concrete is aesthetically unacceptable in certain architectural applications. In the opposite manner, the RGD concrete encompasses pleasant reddish colours. Moreover, the cast cube and mini-beam samples showed excellent surface finish indicating good fresh properties as shown in Fig. 7.23.

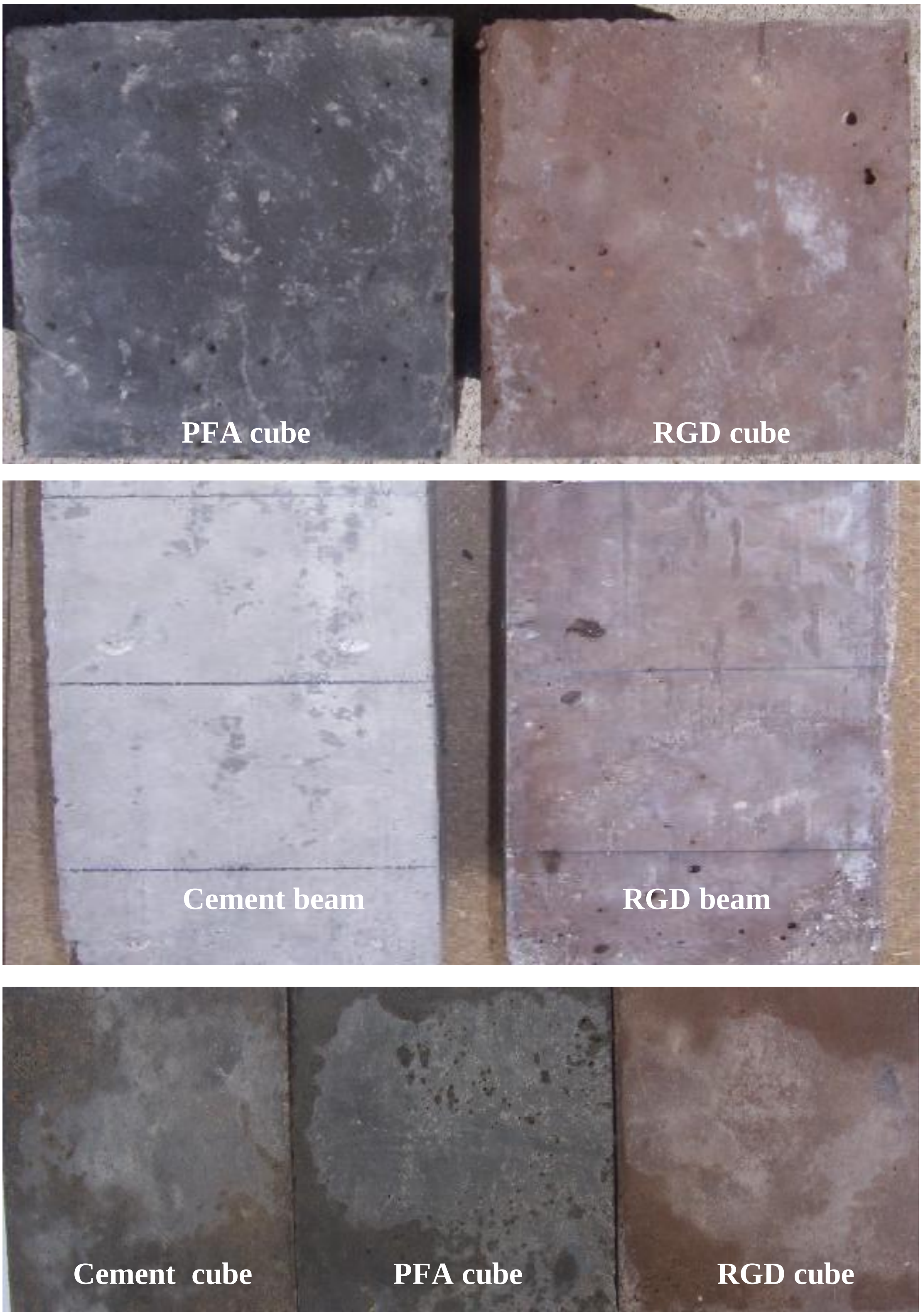


Fig. 7.23 Surface finish of RGD compared to cement and PFA concrete

(iii) Compressive, tensile, and flexural strength of concrete with 30% RGD replacing cement

RGD concrete mixes (Mixes 9 to 12) were prepared as PFA mixes (the only change was RGD instead of PFA). Mixes were tested for the mechanical properties of concrete and the results are presented in Tables 7.28 and 7.29. Data for reference mixes (without RGD) were also listed for comparative purpose. Fracture surfaces of concrete samples with 30% RGD substitution are shown in Fig. 7.24.

Table 7.28 Compressive strength of mixes with 30% RGD replacing cement and reference mixes

Mix	Code	RGD (%)	Compressive strength (N/mm^2)		
			28 days	56 days	90 days
1	NAC-CM	0	69.0 (1.00)	71.2 (1.00)	76.0 (1.00)
2	RAC-CM	0	52.0 (0.75)	58.0 (0.81)	58.3 (0.78)
5	NAC-16WR-0.8SP	0	74.0 (1.07)	89.7(1.26)	91.7 (1.21)
6	RAC-16WR-0.8SP	0	64.0 (0.93)	72.0 (1.01)	73.0 (0.96)
13	NAC-30RGD	30	64.1 (0.93)	62.0 (0.87)	72.6 (0.96)
14	RAC-30RGD	30	46.0 (0.67)	53.0 (0.75)	53.5 (0.70)
15	NAC-16WR-0.8SP-30RGD	30	72.0 (1.04)	76.3 (1.07)	73.8 (0.97)
16	RAC-16WR-0.8SP-30RGD	30	53.6 (0.78)	54.0 (0.76)	54.5 (0.72)

Value in brackets is the ratio of the mix strength at the specified age to the corresponding strength of the NAC control mix. Mix 15 and 16 were also tested at 7 days to check the early age strength of concrete with RGD; 66.4 and 41.5 N/mm^2 were measured respectively.

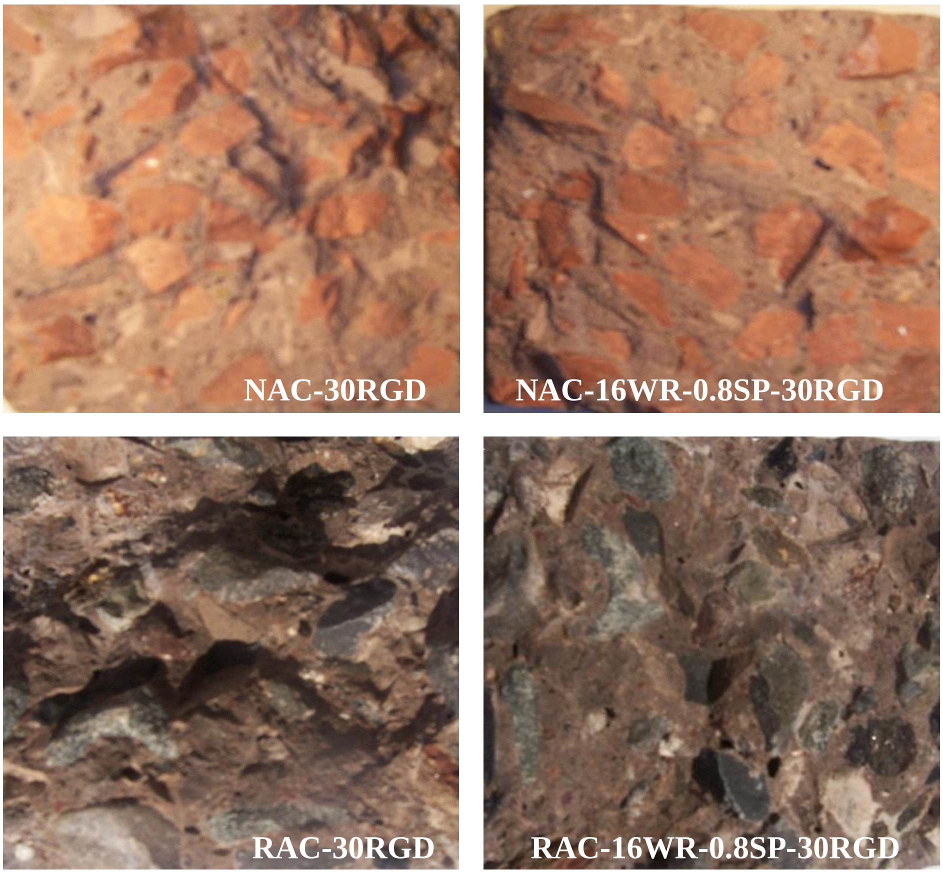


Fig. 7.24 Fracture surfaces of concrete samples with 30% RGD substitution

Table 7.29 Tensile splitting and flexural strength of mixes with 30% RGD replacing cement and reference mixes

Mix	Code	RGD (%)	Splitting tensile strength after: (N/mm^2)			Flexural strength after: (N/mm^2)		$F_{t\,28}/F_{cu\,28}$* (%)
			28 d	56 d	90 d	28 d	56 d	
1	NAC-CM	0	4.7	3.8	4.6	8.10	9.75	6.8
2	RAC-CM	0	3.6	4.1	3.6	7.80	7.87	6.9
5	NAC-16WR-0.8SP	0	5.8	7.2	7.1	11.3	11.3	7.8
6	RAC-16WR-0.8SP	0	4.8	4.4	4.9	10.7	8.20	7.5
13	NAC-30RGD	30	4.8	3.6	4.7	6.15	6.80	7.5
14	RAC-30RGD	30	3.5	3.5	3.7	7.54	5.45	7.6
15	NAC-16WR-0.8SP-30RGD	30	4.6	4.5	5.4	7.56	6.73	6.4
16	RAC-16WR-0.8SP-30RGD	30	3.5	4.6	5.2	7.30	7.02	6.6

*$F_{t\,28}$ and $F_{cu\,28}$ are 28 days tensile and compressive strength respectively

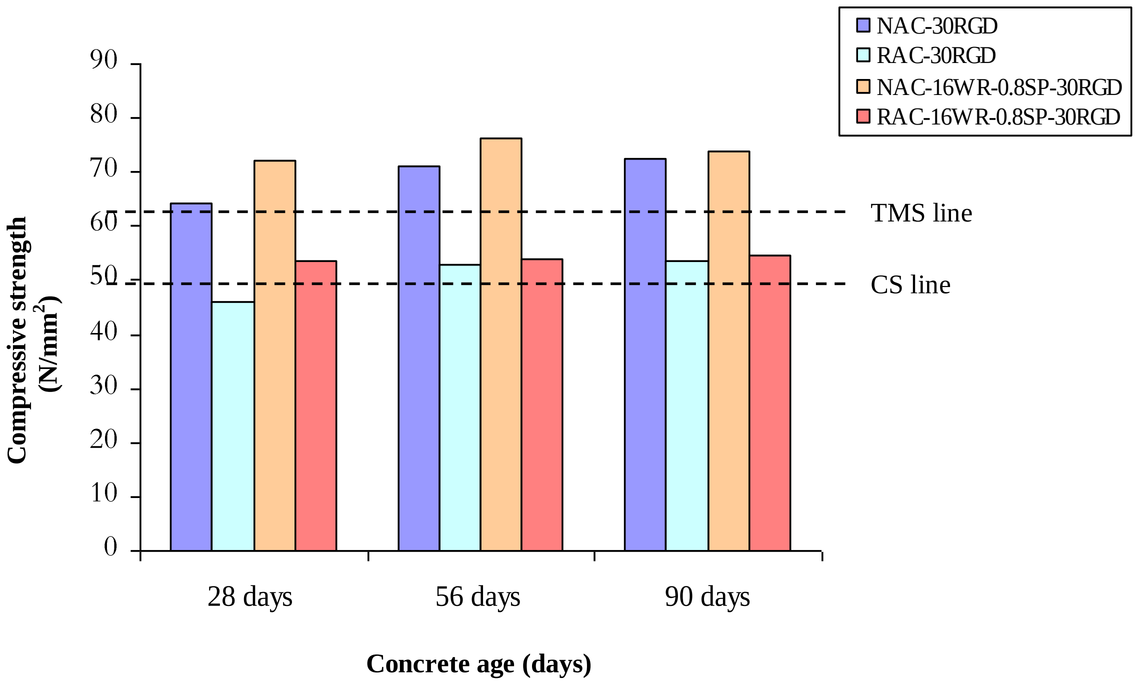


Fig. 7.25 Comparison of compressive strength of mixes with RGD at different ages

Results presented in Tables 7.28 and 7.29 showed that NAC concrete made with 30% RGD substituting cement achieved the target mean strength, *i.e.* 63 N/mm^2. This concrete, which can be regarded as a recycled concrete, attained compressive and tensile strength values better than the reference mix, and comparable flexural strength. Table 7.29 also gives the compressive strength expressed as fractions of that of the standard mix; the ratio ranges from 0.87 to 1.07 with most of the values being above 0.95 for NAC mixes, and from 0.67 to 0.78 with the majority of values above 0.72 for RAC mixes.

RAC mixes with 30% RGD were able to achieve the characteristic strength only after 56 days and were unable to attain the target mean strength at any age unless a proper SP is used at lower w/c. The other noticeable behaviour is that the SP does not improve the strength of RGD concrete mixes that much; a comparison of mixes with and without SP shows small increases in general, and the differences were decreasing with time. However although RAC mixes with 30% RGD achieved acceptable strength (53 N/mm^2) after 56 days, that nevertheless may be employed for different purposes, but it is clear that when two recycled materials (*i.e.* recycled aggregate and RGD) are combined, the resulting concrete is unlikely to achieve the target strength even with the use of SP.

The development of compressive strength of concrete mixes made with 30% RGD at ages of 28, 56 and 90 days is shown in Fig. 7.26.

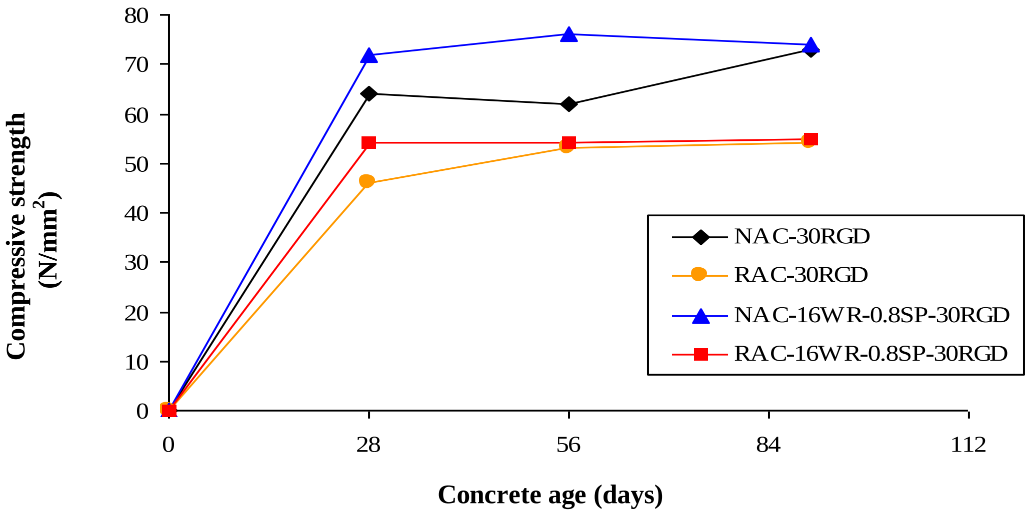


Fig. 7.26 Development of compressive strength of concrete mixes made with 30% RGD

It is remarkable that NAC and RAC concrete mixes made with 30% RGD replacing cement with or without SP end up with almost equal strength after a period of time. Therefore, the superior strength increase of concrete mixes with SP over mixes without SP is more pronounced during the period up to 90 days age for NAC concrete and before 56 days for RAC concrete; beyond these times this difference vanished.

Table 7.30 Relative compressive strength of concrete with 30% RGD

Mix	Code	RGD (%)	Relative compressive * strength after: (%)		
			28 days	56 days	90 days
1	NAC-CM	0	69	103	110
2	RAC-CM	0	52	112	112
5	NAC-16WR-0.8SP	0	74	121	124
6	RAC-16WR-0.8SP	0	64	113	114
13	NAC-30RGD	30	64	97	113
14	RAC-30RGD	30	46	115	116
15	NAC-16WR-0.8SP-30RGD	30	72	106	103
16	RAC-16WR-0.8SP-30RGD	30	54	101	102

*Relative strength is the ratio of the mix strength to corresponding 28 day strength.

Table 7.33 indicates that the improvement of compressive strength of mixes without RGD varies from 10% to 24% and 2% to 16% for mixes incorporating 30% RGD, however, the difference is not significant.

Data in Table 7.29 show that RGD exhibited the same trend as PFA mixes, the ratio of the tensile splitting strength to the compressive strength was consistent at between 6 to 8% for NAC and RAC concrete mixes. This ratio was not shown to be influenced by the incorporation of RGD powder in any mix.

A sample of three cubes of Mix 15 (NAC-16WR-0.8SP-30RGD) was tested to measure the early age strength; a compressive strength of 67 N/mm^2 was obtained at 7 days and 73.8 N/mm^2 at 90 days. A similar mix produced with PFA achieved 50 and 90 N/mm^2 at 7 and 90 days respectively. This result indicates improved early age strength of RGD mixes as compared with similar PFA mixes. However, this is very important in some practical situations when high early age strength is required for quick construction, or rapid dismantling of forms.

(iv) Modulus of elasticity of concrete made with 30% RGD

The static modulus of elasticity (E_s) for concrete mixes made with RGD was determined in a similar way followed for PFA mixes; two samples for NAC and RAC concrete are shown below.

Table 7.31 Modulus of elasticity measurements for NAC-30RGD

Vertical stress (N/mm^2)	Vertical strain	Horizontal strain
0	0	0
137	6	-45
172	8	-73
219	10	-92
268	12	-111
321	14	-137
378	16	-157
438	18	-180
492	20	-203
554	22	-226
588	24	-249
621	26	-264
556	28	-276

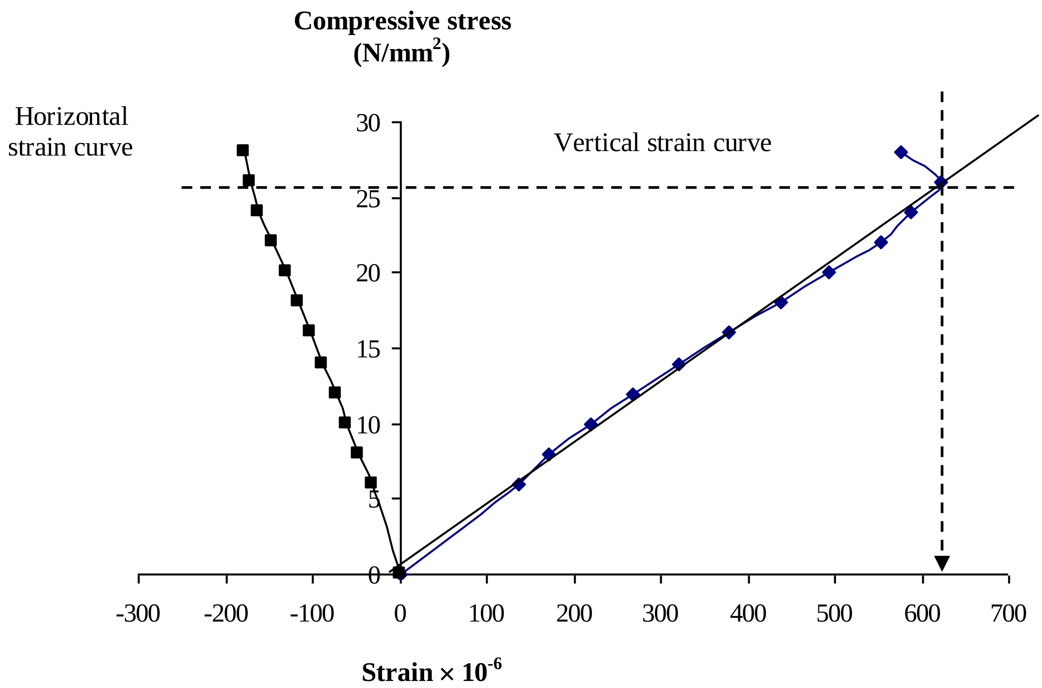


Fig. 7.27 Stress-strain curve for RGD specimen (NAC-30RGD)

Table 7.32 Modulus of elasticity measurements for RAC-30RGD

Vertical stress (N/mm^2)	Vertical strain	Horizontal strain
125	0	-30
186	4	-52
266	6	-83
346	8	-101
430	10	-115
511	12	-132
585	14	-169
648	16	-196
713	18	-218

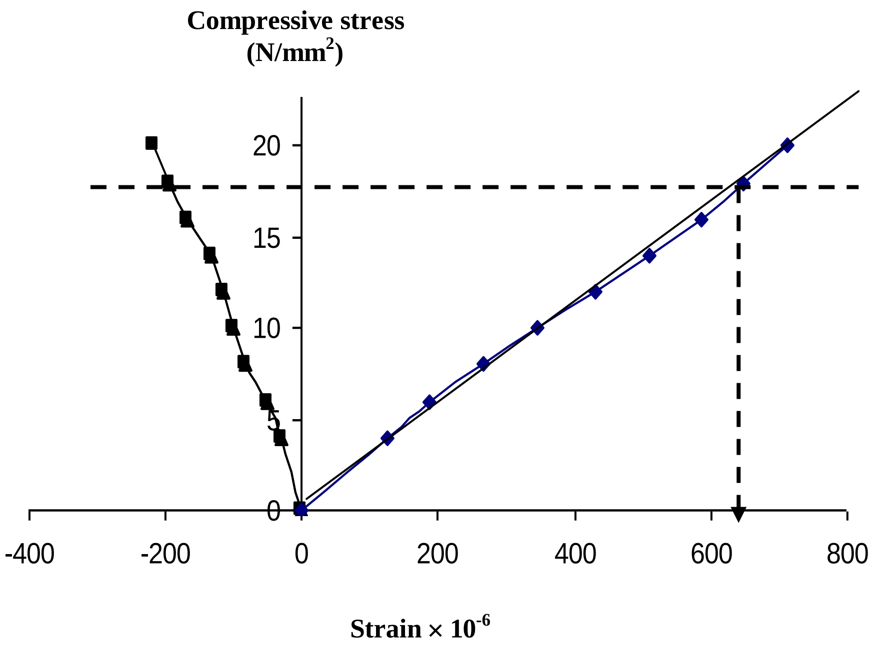


Fig. 7.28 Stress-strain curve for RGD specimen (RAC-30RGD)

Data for all mixes are summarised in Table 7.33.

Table 7.33 Modulus of elasticity of mixes with RGD replacing cement

Mix	Code	RGD (%)	F_{cu} (N/mm^2)	Modulus of elasticity E_s (GPa)	Poisson's ratio	F_{cu} ratio	E_s ratio
1	NAC-CM	0	69	36.3	0.10	100	100
2	RAC-CM	0	52	28.3	0.28	75	78
5	NAC-16WR-0.8SP	0	74	51.4	0.39	107	142
6	RAC-16WR-0.8SP	0	64	40.0	0.30	93	110
13	NAC-30RGD	30	64	41.4	0.25	93	114
14	RAC-30RGD	30	46	28.3	0.25	67	78
15	NAC-16WR-0.8SP-30RGD	30	72	49.7	0.27	104	137
16	RAC-16WR-0.8SP-30RGD	30	54	39.0	0.25	78	108

E_s ratio is the percentage ratio of the mix modulus of elasticity at 28 days to the corresponding value of the NAC control mix, and similarly for the F_{cu} ratio.

Results presented in Table 7.33 show that E_s tended to improve with the addition of RGD and SP, particularly with NAC mixtures; NAC modulus increased by 14-37%, most likely due to greater stiffness of NA and a strong matrix. NAC with 30% RGD and SP achieved the highest E_s value. In contrast, only superplasticized RAC has shown to improve by 8% due to decreased strength of RAC without SP. The unsuperplasticized RAC Mix 14 (with only RGD) produced an elastic modulus equal to that of RAC standard mix; while the RAC mix with SP attained the highest modulus of 39 GPa ; which is indeed better than the NAC standard mix. However, although the moduli attained by RAC are in fact lower than those offered by NAC, they remain good with respect to their stiffness for such a material.

Slender sections need to attain high strength and elastic modulus; such sections are preferable for cost and space saving. In this context, it has been found earlier that the density of concrete decreased with the addition of 30% RGD which implies lower dead weight, and if this is combined with the findings of the preceding section, namely the increase of strength and elastic modulus at the same level of replacement (30%), then this should be considered as a credit to this material.

(d) Comparison between PFA and RGD concrete

This paragraph was prepared just to compare PFA and RGD concrete; it may worth reminding the reader that RGD and PFA mixes were produced with similar proportions, materials, and replacement levels.

(i) Comparison between PFA and RGD concrete on the basis of strengths and elasticity modulus

For comparative purposes Tables 7.34 to 7.36 and Figs 7.25 to 7.28 were prepared.

Table 7.34 Compressive, tensile and flexural strength and modulus of elasticity of concrete made with 30% PFA or 30% RGD

Mix	Code	Compressive strength (N/mm^2)	Tensile strength (N/mm^2)	Flexural strength (N/mm^2)	Modulus of elasticity (GPa)
9	NAC-30PFA	65	3.43	7.52	39.50
13	NAC-30RGD	64	4.83	6.15	41.35
10	RAC-30PFA	51	3.80	7.16	24.93
14	RAC-30RGD	46	3.50	7.54	28.31
11	NAC-16WR-0.8SP-30PFA	75	4.20	10.6	50.69
15	NAC-16WR-0.8SP-30RGD	72	4.59	7.56	49.65
12	RAC-16WR-0.8SP-30PFA	62	4.12	7.40	37.16
16	RAC-16WR-0.8SP-30RGD	54	3.48	7.30	39.00

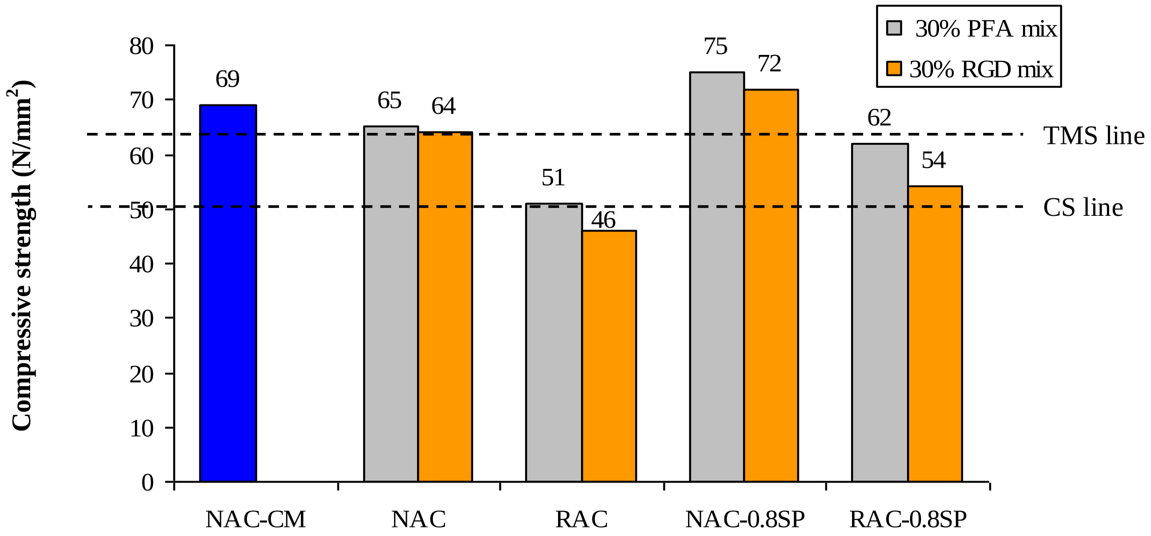


Fig. 7.29 Comparison of 28 day compressive strength of mixes with PFA and RGD

With respect to compressive strength, the data in Fig. 7.30 show that concrete with 30% PFA marginally better than similar concrete made with 30% RGD.

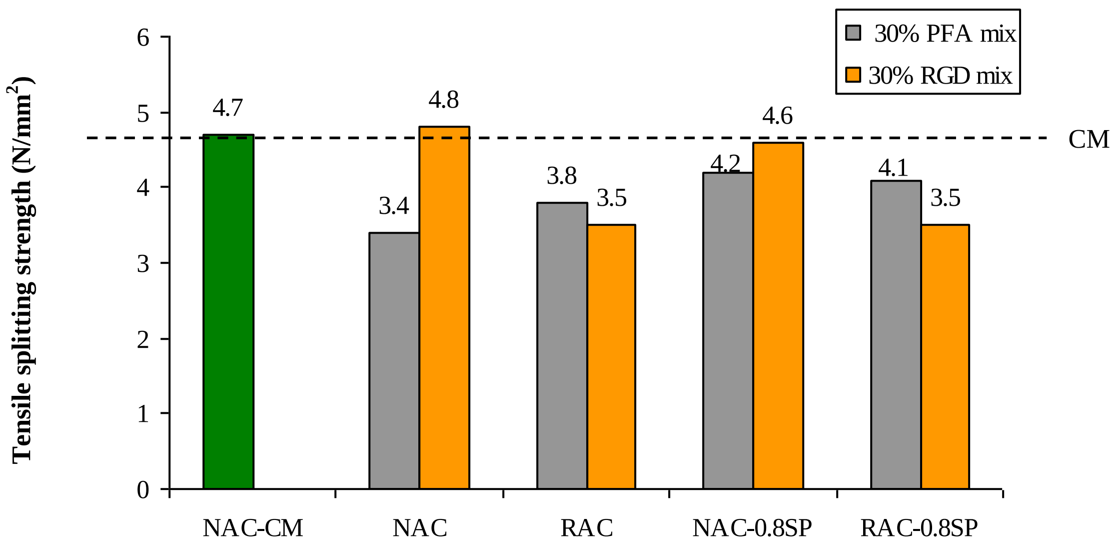


Fig. 7.30 Comparison of tensile splitting strength of mixes with PFA and RGD and the 28 day control mix

With respect to splitting strength, as can be seen in Fig. 7.30 both PFA and RGD concrete are generally slightly below that produced by the reference mix; however NAC concrete with RGD is better than that with PFA. Similarly, for the flexural strength, both PFA and RGD concrete mixes are in general below that produced by the standard mix (see Fig. 7.31). That is more pronounced in RAC mixes.

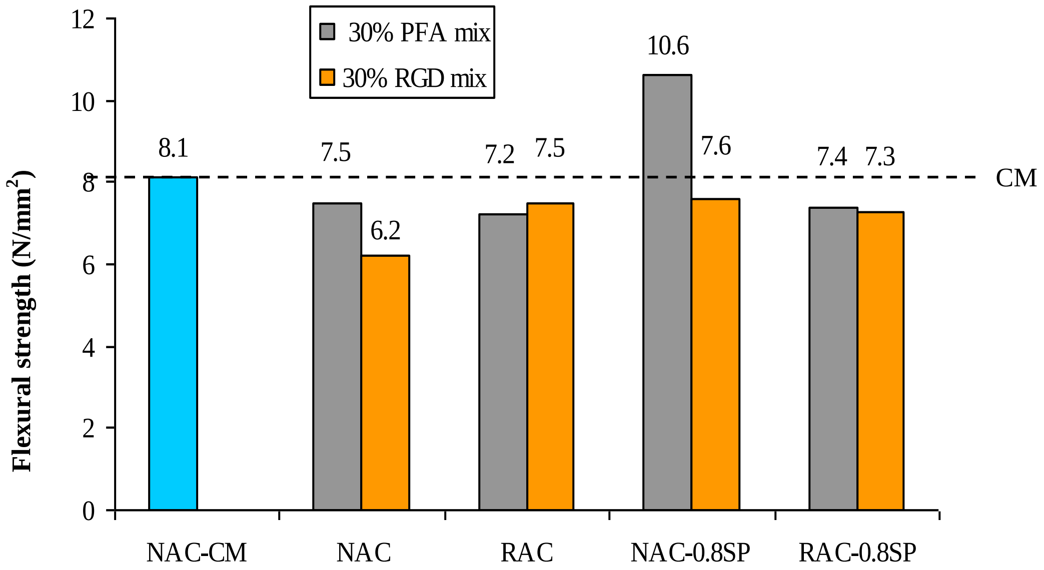


Fig. 7.31 Comparison of flexural strength of mixes with PFA and RGD and the 28 day control mix

For the elastic modulus Fig. 7.32 indicates that E_s of the superplasticized concrete with RGD was almost equal to or slightly better than, that of the PFA concrete. E_s for mixes made with RAC without SP is less than that of similar NAC and the reference mix. However the measured values are not very low when compared to those observed in ordinary concrete. The higher modulus of elasticity of the superplasticized mixes is attributed to the improved matrix stiffness and mortar-aggregate bond.

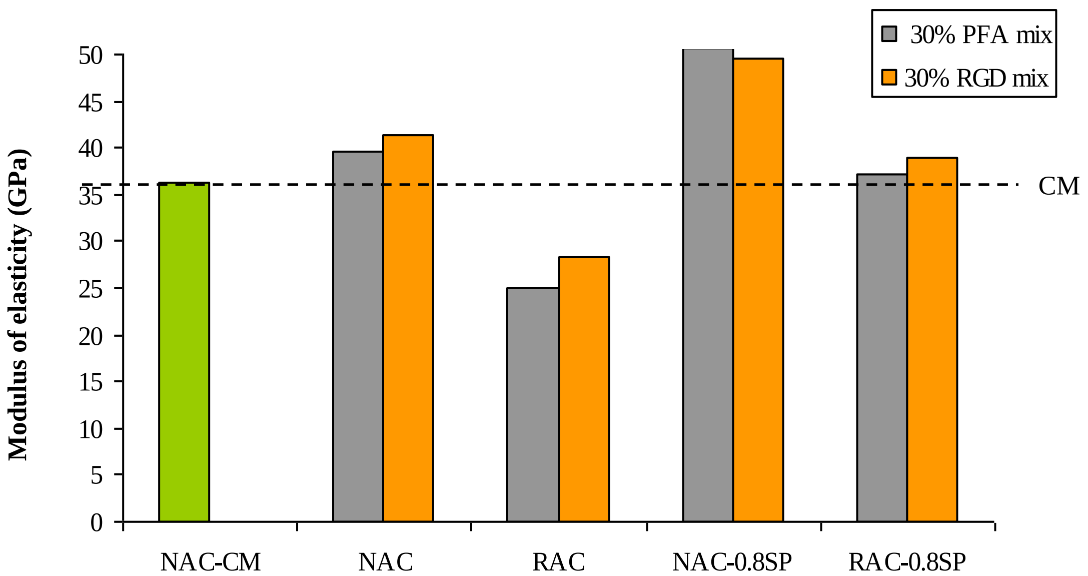


Fig. 7.32 Comparison of modulus of elasticity of mixes with PFA and RGD and the 28 day control mix

Referring to the chemical composition of PFA and RGD given in Chapter 4, similarities in the major components forming the two materials can be seen in Table 7.35; evidently, RGD contains more silicon dioxide, the material believed to be the most important factor in the PFA pozzolanic reaction kinetics. The corresponding average percentages in ground blastfurnace slag (GBFS) which is also used as a substitute for cement in concrete mixes are listed here for comparison.

Table 7.35 Major components in PFA, RGD and GBFS

Component	Percentage in PFA	Percentage in RGD	Percentage in GBFS
Silica (SiO_2)	51	61.4	37
Alumina (Al_2O_3)	25	16.3	10
Ferric oxide (Fe_2O_3)	7.5	3.6	0.75

To make a comparison between the influence of PFA and RGD on the compressive strength of concrete at early (28 days) and later ages (90 days), Table 7.36 was prepared.

Table 7.36 Comparison of strength of concrete made with 30% PFA and 30% RGD

<table>
<tr><th>Mix</th><th>Code</th><th>28 day F_{cu} (N/mm²)</th><th>28 day strength ratio (R₂₈)</th><th>90 day F_{cu} (N/mm²)</th><th>90 day strength ratio (R₉₀)</th><th>Relative F_{cu} (%)</th></tr>
<tr><td>9</td><td>NAC-30PFA</td><td>65</td><td rowspan="2">0.99</td><td>76</td><td rowspan="2">0.96</td><td>116</td></tr>
<tr><td>13</td><td>NAC-30RGD</td><td>64</td><td>73</td><td>113</td></tr>
<tr><td>10</td><td>RAC-30PFA</td><td>51</td><td rowspan="2">0.91</td><td>60</td><td rowspan="2">0.99</td><td>117</td></tr>
<tr><td>14</td><td>RAC-30RGD</td><td>46</td><td>54</td><td>116</td></tr>
<tr><td>11</td><td>NAC-16WR-0.8SP-30PFA</td><td>75</td><td rowspan="2">0.95</td><td>90</td><td rowspan="2">0.82</td><td>119</td></tr>
<tr><td>15</td><td>NAC-16WR-0.8SP-30RGD</td><td>72</td><td>74</td><td>102.5</td></tr>
<tr><td>12</td><td>RAC-16WR-0.8SP-30PFA</td><td>62</td><td rowspan="2">0.87</td><td>69</td><td rowspan="2">0.79</td><td>112</td></tr>
<tr><td>16</td><td>RAC-16WR-0.8SP-30RGD</td><td>54</td><td>55</td><td>102</td></tr>
</table>

R_{28} is the ratio of the 28 day compressive strength (F_{cu}) of the mix with RGD to the strength of the corresponding mix with PFA, similarly $R_{90.}$ Relative compressive strength is the percentage ratio of the 90 day strength to the corresponding 28 day strength.

Inspection of Table 7.36 reveals that the relative compressive strength ranged from 0.87 to 0.99 and from 0.79 to 0.99 at 28 days and 90 days respectively. This ratio is influenced by the aggregate type as its being smaller for RAC and larger for NAC. With respect to the development of strength, the increase was 12 to 19% for mixes made with PFA and 2 to 16% for mixes with RGD. This suggests that the compressive strength was developing at a similar rate. However, despite chemical analysis showing that RGD contains more silicon dioxide (SiO_2) than PFA; the constituent accountable for pozzolanic reaction (more details were given in Chapters 2 and 3), results of mixes with RGD did not show any extraordinary contribution to strength over time, that may be attributed to the passive chemical nature of silica. Results are in agreement with (Illston

& Domone 2001) who reported that the key to the pozzolanic behaviour is the structure of the silica which must be in glassy or amorphous form with a disordered structure, which is formed in rapid cooling from a molten state. A uniform crystalline structure which is formed in a slower cooling, such as is found in silica sand is not chemically active.

Another observation worthy of mention is that concretes with low and higher levels of RGD cement replacements, and those with PFA have exhibited similar behaviour. Both materials have caused a decrease of compressive and tensile strengths and concrete stiffness.

It can be concluded that considerable amounts of RGD are likely to be created as a by-product of stone crushing in the aggregate industry. RGD nowadays is available at effectively no cost other than its transportation cost and indeed presents a problem from the view point of disposal, environmental pollution, and potential health hazard. This part of the study showed that RGD can be successfully recycled in concrete, predominantly in NAC mixes thereby converting it to an eco-efficient material. It can be utilised with the help of SP as a partial substitute for cement to produce recycled concrete without affecting strengths and stiffness. However, a substitution level up to 30% RGD is suggested as it was proven to produce strengths better than the values exhibited by the reference mix. A cutback of 30% of the cement content represents a very good cost saving in addition to other technical and environmental benefits of utilising this by-product in concrete. The use of RA to produce concrete with RGD is however also possible for medium strength concrete, but is not recommended for structural concrete. Moreover, lower strength RAC produced either without admixtures or with SP, PFA, and RGD is still exploitable in certain civil engineering applications, especially in non-structural concrete where lower strength is required.

(ii) Comparison of the measured strength and estimated mechanical properties of concrete made by PFA and RGD

As introduced for PFA mixes, the elastic modulus is ideally measured directly on concrete specimens, but from an experimental point of view, this is not always simple compared, for instance, to the measurement of compressive strength. Tensile and flexural strength and the elastic modulus of concrete are often estimated using either theoretical or empirical approaches. In latter case, also the most widely used, various concrete properties are usually expressed as functions of compressive strength. On the other hand, several national codes suggested various formulas for normal and high strength concrete. Some models used in this perspective for NA and RA concretes are presented in Table 7.37.

Table 7.37 Models used to estimate concrete properties from the compressive strength

Property	Reference	Used for	Model
Modulus of elasticity E_s (N/mm^2)	ACI-318-2002	Normal NAC	$4127\sqrt{F_{cu}}$
	CEB-FIP 1990	Normal NAC	$(21.5)\ (\sqrt[3]{F_{cu}/10})$
	GB50010-2002	Normal NAC	$(10^5) / (2.2+ 34.7/F_{cu})$
	Dhir *et al.*	RAC	$370F_{cu} + 13100$
	Ravindrajah *et al.*	RAC	$7770(F_{cu})^{0.33}$
	Dillmann	RAC	$634.43\ F_{cu} +3057.6$
	Xiao *et al.*	RAC	$(10^5) / (2.8+ 40.1/F_{cu})$
Modulus of rupture F_r (N/mm^2)	ACI-318-2000	Normal NAC	$0.54\sqrt{F_{cu}}$
	CEB-FIP 1990	Normal NAC	$0.81\sqrt{F_{cu}}$
	Xiao *et al.*	RAC	$0.75\sqrt{F_{cu}}$
Tensile strength F_t (N/mm^2)	ACI-318-2000	Normal NAC	$0.49\sqrt{F_{cu}}$
	CEB-FIP 1990	Normal NAC	$1.4(F_{cs}/10)^{0.67}$
	GB50010-2002	Normal NAC	$0.19\ (F_{cu})^{0.75}$
	Xiao *et al.*	RAC	$0.24(F_{cu})^{0.65}$

ACI-318-2002: American Concrete Institute Committee for Structural Concrete
CEB-FIP 1990: Comité Euro-International du Béton- Model Code 1990
GB50010-2002: Chinese code for Design of Concrete Structures

The purpose of the analysis carried out here is to check the conformity of the measured properties to those estimated. The estimated properties by the corresponding method listed in Table 7.37 are arranged in Tables 7.38 to 7.41.

Table 7.38 Modulus of elasticity E_s (GPa) obtained by different models and the measured E_s for NAC mixes made with PFA and RGD

Mix	Code	F_{cu} (N/mm^2)	E_s (ACI-318)	E_s (CEB-FIP)	E_s (GB-50010)	E_s Measured
9	NAC-30PFA	65	33.27	40.12	36.58	39.50
13	NAC-30RGD	64	33.04	39.94	36.48	41.35
11	NAC-16WR-0.8SP-30PFA	75	35.84	42.16	37.59	50.69
15	NAC-16WR-0.8SP-30RGD	72	35.02	41.52	37.29	49.65

Table 7.39 Modulus of elasticity E_s (GPa) obtained by different model and the measured E_s for RAC mixes

Mix	Code	F_{CU} (N/mm^2)	E_s (Dhir *et al.*)	E_s (Ravindrajah *et al.*)	E_s (Xiao *et al.*)	E_s Measured
10	RAC-30PFA	51	31.90	28.78	27.86	24.93
14	RAC-30RGD	46	30.12	27.84	27.24	28.31
12	RAC-16WR-0.8SP-30PFA	62	35.89	30.69	28.98	37.16
16	RAC-16WR-0.8SP-30RGD	54	32.93	29.30	28.18	39.00

Table 7.40 Modulus of rupture (F_r) obtained by different model and the measured F_r for NAC mixes

Mix	Code	F_{cu} (N/mm^2)	F_r (ACI-318)	F_r (CEB-FIP)	F_r Measured
9	NAC-30PFA	65	4.35	6.53	7.52
13	NAC-30RGD	64	4.32	6.49	6.15
11	NAC-16WR-0.8SP-30PFA	75	4.69	7.03	10.6
15	NAC-16WR-0.8SP-30RGD	72	4.58	6.87	7.56

Table 7.41 Modulus of rupture (F_r) obtained by different model and the measured F_r for RAC mixes

Mix	Code	F_{cu} (N/mm^2)	F_r (ACI-318)	F_r (CEB-FIP)	F_r (Xiao *et al.*)	F_r Measured
10	RAC-30PFA	50.8	3.85	5.77	5.35	7.16
14	RAC-30RGD	46	3.66	5.49	5.10	7.54
12	RAC-16WR-0.8SP-30PFA	61.6	4.24	6.36	5.89	7.40
16	RAC-16WR-0.8SP-30RGD	53.6	3.95	5.93	5.50	7.30

Results presented in Tables 7.38 to 7.41 showed that the existing relationship of elasticity modulus, modulus of rupture, and tensile strength for normal and recycled aggregate are not valid for NAC and RAC concrete made with chemical and/or mineral admixtures. Perhaps such estimators were developed for ordinary concrete in which minerals and chemical admixtures were not used. To develop new prediction models, incorporating admixtures, a database or copious experimental data are needed. Therefore, in view of the absence of such results, which can be considered a basis for possible statistical analysis, to derive appropriate interrelationships of the mechanical properties for this type of concrete, it is recommend that the properties of RAC concrete incorporating cementitious material and/or chemical admixtures should be measured on a case-by-case basis. However other influencing factors supporting this conclusion could be cited in addition to the large variety of aggregate combinations; the most important are the regional differences, the production method, the curing, the testing method, and the specific standard used (Xiao *et al.* 2006).

7.4 SUMMARY

From this chapter, the following are the major conclusions:

1. The effect of blending of RA and NA was a significant influence on the mechanical properties of concrete mixtures tested. Replacement of 50% of the NA by RA has resulted in considerable improvement of the mechanical

performance of the superplasticized RAC.

2. For the selection of the best proportioning option of concrete mixes, results showed that the compressive strength should not be considered as the only decisive factor; various fresh properties and the internal structure of concrete are equally important.

3. The incorporation of 30% of cement by PFA in NA as well as RA concrete mixes improve the workability of concrete; in presence of SP, the slump of PFA concrete was increasing.

4. The compressive strength of the superplasticized RAC with 30% PFA was much better than the RAC standard mix (improved by 33%) and similar to that of the NAC reference mix at 90 days. However, strength at 28 days was marginally below the target mean. RAC strengths within the range of 50-62 N/mm^2 at 28 days, 52-67 N/mm^2 after 56 days and 59-69 N/mm^2 after 90 days were achieved; these ranges practically cover the strength commonly required for several engineering applications.

5. Results showed that NAC and RAC concrete mixes with 30% PFA as a substitute for the cement exhibited substantial increase of strength at later ages; 7 day compressive strengths improved by 80% and by about 90% for NAC and RAC concrete respectively. The enhancement was mainly due to continued hydration of cement and the pozzolanic nature of PFA. Further improvement is therefore possible later on. A mild adverse influence of PFA on the tensile and flexural strength was observed.

6. The SP is an essential component in producing good quality RAC incorporating PFA.

7. The ratio of the tensile splitting strength to the compressive strength of RAC concrete with PFA was measured as 6 to 8%; which is a little lower than that usually attained from normal strength concrete without additives (10% in average).

8. In agreement with several previous studies, results of this investigation showed that for all NAC and RAC mixes with 30% PFA, the stress-strain relationships were non-linear.

9. In agreement with previous findings, the modulus of elasticity of concrete was influenced by the aggregate strength, and in turn by the concrete strength, in addition to the presence of PFA and SP in the mix. Results showed that E_s values, for both types of concrete, were generally increasing with the increase in compressive strength. The performance characteristics of the superplasticized RAC concrete with PFA (RAC-16WR-0.8SP-30PFA) was better than the NAC control mix (NAC-CM).

10. The commonly used equations for relating the mechanical properties of ordinary concrete cannot be applied for NAC and RAC produced with chemical or mineral admixtures. Design or quality control parameters need to be obtained by direct measurement.

11. Considerable amounts of RGD powder estimated as 1-2% of the total crushed aggregate are produced in aggregate quarries. In addition to space problems, RGD presents health risks to the staff and local residents, incurs costly disposal charges, increases the size of landfill disposal, and impairs the environment. The large-scale utilisation of this material in producing beneficial products could minimise or eliminate these unpleasant consequences.

12. Physical and chemical properties of RGD powder showed, to a large extent, similarity to PFA, and therefore concrete mixes with RGD and PFA can be designed in the same way.

13. Despite the higher percentage silicon dioxide content of RGD powder, which was estimated as 61% when compared with PFA silica content of 51%, but results showed RGD is a less reactive material than PFA.

14. Experimentally determined mechanical properties of RAC should be directly measured on representative samples; the estimation by numerical formulae could lead to wide variations.

15. The results showed that NAC produced with RGD at cement replacement level of 30% exhibited strength values either comparable or better than those of the reference mix. Strength values are marginally less than those of similar concrete made with PFA, but an equal or improved modulus of elasticity ensured. In the same way, RAC with RGD showed strengths comparable to equivalent concrete with PFA, but a larger elastic modulus.

16. Concrete mixes containing 30% RGD powder showed good fresh properties, better than expected mechanical properties, and excellent surface finish. The concrete produced covers a wide range of the design strengths typically required in practice.

17. Concrete with 30% RGD demonstrated better early age strength than similar concrete with PFA. This however could address the problems associated with the early strength of PFA concrete. Any RGD addition higher than this amount resulted in a decrease of strength, although it remained acceptable at the 50% level.

18. Up to a certain level (30 - 40% of cement), RGD can be recycled in concrete; it offered good fresh and hardened properties, cut the amount of cement, therefore reduced the pressure on natural resources and curtailed the risks inherent to aggregate production.

CHAPTER 8
RECYCLED SELF-COMPACTING CONCRTE
MADE WITH PFA OR RGD

8.1 INTRODUCTION

The name of this type of concrete *i.e.* self-compacting concrete (SCC) implies, *per se*, the key feature of this technology. SCC does not require vibration for placing, and, due to its high flowability, can simply be poured into concrete formworks and achieve full compaction under its own weight causing no segregation or bleeding. Hardened properties are not much different to conventional concrete; therefore SCC is generally regarded as an extension of normal high-consistency concrete. SCC is a relatively new product to the construction industry; it was developed in Japan in the late 1980s, mainly due to the lack of skilled workers, and subsequently spread to Europe; particularly the Scandinavian region, and more recently North and South America. However, SCC nowadays is widely used worldwide.

For a concrete to pass as a self-compacting concrete, it must satisfy certain requirements set out in BS EN 206-9: Additional Rules for Self-compacting Concrete (SCC); testing methods and standards are covered in BS EN 12359 *:* Testing fresh concrete, parts 8 to 12. In this chapter, brief details of SCC will be given; many more elaborate publications are available elsewhere. In this investigation, an attempt will be made to produce SCC using recycled materials and industrial by-products. RA, PFA, and RGD will be tried. The information re-introduced here is only that perceived essential for successful creation of this new product.

8.2 OBJECTIVES OF THE INVESTIGATION

Literature showed that different natural aggregates have been successfully used to produce SCC, but little data are available on SCC made with RAC, and almost no data on SCC made with the combination of recycled aggregates and industrial by-products. The main objective of this part of the study therefore was to test out if the coarse RA and industrial by-products such as PFA and RGD can, or otherwise not, be used for high slump concrete *i.e.* SCC. If self-compacting recycled aggregate concrete (RAC-SCC) is shown to achieve similar properties as its counterpart natural aggregate concrete (NAC-SCC) then it can be a viable alternative for the concrete industry.

8.3 SCC: A REVIEW

(a) Published literature

Due to the very limited data on the use of recycled materials in SCC, a very brief review of general aspects of NAC-SCC will be given. As mentioned, SCC has a short history as a product, but literature showed there has been a wide range of applications. In this section, a number of previous research papers were reviewed (principally Higuchi 1998; Okamura & Ouchi 1999; Domone & Jin 1999 & Domone 2006; Noguchi & Tomosawa 1999; Saak & Shaf 1999; Ferraris *et al.* 2000; Su *et al.* 2001; Ho *et al.* 2002; Felekoglu & Baradan 2003 & Felekoglu *et al.* 2006; Hughes 2002; Topcu & Ugurlu 2003; Xie *et al*. 2002; Zhu & Gibbs 2005; Mustafa *et al.* 2006; Burak 2007; Nuno *et al.* 2007; and Tayyeb *et al.* 2009). The review was not focused on each individual publication and a general summary of the major themes will be provided instead. The volume of research in this topic has been progressively increasing and spreading worldwide. Several published case studies, which include detailed information on the selection of component materials, mix proportions, and the resulting concrete properties, are also available. The findings of the previous investigations were considered an essential prerequisite for understanding the characteristics of SCC and successful production of RAC-SCC. The key points drawn are summarised in the following paragraphs.

Some of the published data paid attention mainly to the technical advantages and disadvantages of SCC compared to ordinary vibrated concrete, the reason for using SCC in a particular application, its impact on overall cost, construction time, and health and safety. Most of the previous research papers were in agreement on the issue of the mechanical properties; data showed that SCC exhibited better workability, but similar mechanical properties to conventional concrete.

Reviewed works (research and case histories) showed that when preparing mix proportions of an SCC, it is critical to take to account of the general principles for achieving the required properties. The most important are:

- Lower coarse aggregate content
- Increased paste content
- High powder content
- Low water to powder ratio
- High superplasticizer dosage

- The use of viscosity-modifying agents (although uncommon, the effects are significant).

Literature showed that these requirements are essential to the production of SCC; the low coarse aggregate content is a necessity to increase the concrete's ability to pass through narrow gaps in reinforcement while reducing the risk of aggregate bridging. The low aggregate content, jointly with the high powder content, will result in increased paste content thus providing the required lubrication for the individual coarse aggregate particles and increase the mix's fluidity. Limiting the fine aggregate content and water to powder ratio will lead to increased fluidity. Superplasticizer and viscosity-modifying agents (VMA) (if proven necessary) provide adequate filling ability, provide stability, reduce the risk of segregation and bleeding, and reduce the sensitivity of the mix to variations in material properties during production, particularly the aggregate moisture content. However, (Dinakar *et al.* 2008) reported that some previous studies indicate that the incorporation of fly ash usually eliminates the need for VMA. In addition, VMA admixtures are expensive (15-20% of average concrete cost; about £12-£15 *per* m^3 at the UK's 2009 prices) and can substantially increase the cost of SCC.

With respect to aggregates, data showed that crushed aggregates were more frequently used than natural gravel, whereas vary rarely were lightweight aggregates used; meanwhile the literature did not show any applications of RA in SCC. Sizes in the range of 12-20 mm were often used. Smaller size aggregate is preferable for concrete pumping, while concrete with large-sized aggregates is commonly a first choice for large volumes or mass concrete such as gravity structures and large foundations.

Portland cement is the most commonly used binder component, though other types of cements have also been used. Mineral admixtures such as PFA, slag and silica fume as well as filling powders, such as limestone and chalk, were used for different purposes for instance to reduce heat of hydration, strength decrease or increase, *etc*. Limestone powder was the most often used. Average coarse aggregate contents of SCCs varied from 29-35% by volume of concrete; equivalent to 770-925 kg/m^3 for a typical aggregate relative density of 2.65. Sometimes reaching 40%, it was never more than 45%. The paste content is typically within the range 30-42% by volume of concrete, the average powder content is about 500 kg/m^3 and can be as much as 600 kg/m^3 (Domone 2006).

Most of the studies confirmed that the water to powder ratio has significant influences on both fresh and hardened properties of SCC. The powder composition has a major effect on the hydration process and hence the heat output, thermal stresses, as well as the strength gain; and powders are therefore usually used to control these properties. The water to powder ratio (Domone 2006) is within the range 0.28 to 0.42 and rarely up to 0.48. The fine aggregate content expressed in terms of the mortar composition varies from 40-52%. However, studies showed that powder content and water to powder ratio are less critical for successful SCC compared to coarse aggregate content, fine aggregate percentage in mortar, and the paste content of the concrete therefore concrete producers have greater flexibility with the first two components, *i.e.* the powder content and the water to powder ratio.

In all cases reviewed, a superplasticizer was considered an essential component of the SCC. In many cases, a wide range of dosages was observed; that is maybe due to the variation in the mixture proportions and the desired performance of the end product. Viscosity-modifying agents of various forms were also (sometimes) used; in many cases it was reported that the incorporation of fly ash eliminated the need for VMA.

As regards fresh properties, literatures showed that the slump flow test, which measures the flow capacity of SCC, has been most generally used. Mostly, a slump flow value in the range of 600-750 mm was measured. Flow rate values, expressed as T_{500}; the time during which the released slump will spread to a diameter of 500 mm, were extensively reported, though other means such as V- and O-funnel were also, but less frequently, used. T_{500} were reportedly variable, the range could be 1.8 s to more than 12 s. The slump flow and the rate values together define the filling ability of concrete; however, previous studies did not show a defined relationship between slump flow and flow rate values, indicating no interdependency of these properties. The L-box test is another test used to check the passing ability of the concrete expressed by a ratio known as the passing ability (PA) ratio (see Section 8.4 (b) iii for more details). It determines whether the concrete can flow through confined spaces and reinforcement without causing any blockages or segregation of the aggregate. When the PA ratio is equal to unity, this indicates that all concrete has passed through the reinforcement. Brief details of the tests will be given in this chapter.

With respect to compressive strength, literature showed it was possible to produce SCC with strength to suit nearly all ordinary concreting situations. Strength values ranged from 20 to nearly 100 N/mm^2; with most mixes in excess of 40 N/mm^2. Properly produced SCC suffers no loss of stability, *i.e.* it remains homogenous throughout, with no aggregate segregation as the aggregates stay in suspension despite the higher fluidity of the product. Fresh and hardened strength properties are achieved through the use of increased quantities of cement and fine fillers, as well as concrete superplasticizers.

The overall impression formed from the review is that SCC comprises a large group of mixes, and there is no unique mix for a given application or set of requirements. All requirements and measures necessary for ensuring good quality ordinary concrete are applicable for the counterpart SCC. However, the following advantages can be stated as a result of this general review:

1. SCC is capable of filling all sections of complex shuttering perfectly, therefore will allow the engineer to design more complex connections, and facilitate the process of jointing pre-cast concrete units and cast *in situ* structures.
2. SCC is ideal for deep concrete sections designed with dense reinforcement.
3. The health and safety aspects of self-compacting concrete are also another benefit of the product; SCC avoided the need for a typically noisy piece of vibration equipment (*c.* 95 dBA at 1 m), therefore improving the work environment while reducing both cost and CO_2 emissions. The risk of human injury is reduced as the vibrator is no longer needed for concrete elements that were less accessible.
4. As SCC can be easily pumped into high-rise concrete buildings; the increased volume of concrete will have a positive influence on the construction time and therefore the rate at which the whole structure is erected. In addition it requires fewer workers in comparison to traditional methods.
5. SCC can be a good alternative to ordinary concrete placed in *situ* or for pre-cast production. The early strength gain of SCC, which is attributed to the superplasticizer, guarantees the quick removal of forms for multiple uses.
6. In contrast to ordinary concrete, SCC for the full frame, or a large part of the structure, can be cast in one pour through an appropriate timber or steel mould bolted and sealed, instead of the columns and beams being shuttered independently where joints are usually developed which weaken the structure.

Because of these advantages and maybe others, literature showed that SCC is gradually replacing much conventional concrete. However, the higher initial cost of SCC concrete over conventional concrete has hindered its wider application to general construction. Therefore, the cost needs to be reduced to an acceptable limit; the key factor is to produce SCC concrete, having similar properties, with alternative low-cost materials. In this part of the study RA, PFA, and RGD will be examined.

(b) British Standard and EFNARC guidelines for SCC

(i) British Standards and Concrete Society practice

Because the fresh properties of SCC are very different from those of ordinary concrete, a number of test methods have been developed, BS EN 12359, Parts 8 to 12 present the testing protocols for fresh concrete. Similarly, concrete production and testing are covered in BS EN 206-9: 2007: Additional Rules for SCC. BS EN 12359 states that for a concrete to be classed as self-compacting it must be able to flow and compact under its own weight, completely fill all formwork and surround all reinforcement and ducts yet maintain its homogeneity.

Useful information on aspects of SCC is summarised in two Concrete Society Current Practice Sheets. The first was published in 2001; SCC: the material and its properties, and the second in 2005; SCC: production and use (copies are available on The Concrete Society’s web site www.concrete.org.uk).

(ii) EFNARC Guidelines

In 2002, the European Federation of Specialist Construction Chemicals and Concrete Systems (originally EFNARC, the European Federation of National Association Representing Concrete producers) published the first edition of their ‘Specification and Guidelines for SCC’ which provided state of the art information for both producers and users. The same group published, in May 2005, comprehensive guidelines ‘The European Guidelines for Self-Compacting Concrete’ for its specification, production and use (EFNARC 2002 & 2005). This represents a state of the art document addressed to specifiers, designers, producers, and users of the product. In addition to the definition of technical terms, description of SCC properties and use, the guidelines provide information on standards related to the testing of concrete materials and to testing of concrete itself in its fresh and hardened states.

The EFNARC guidelines were observed to have an increased acceptance by individuals, many research centres, and concrete producers all over the world. They have been cited as a major reference in most recently published work. These guidelines are not in any way a replacement for other concrete standards, current standards related to concrete and its constituents are applicable alongside these guidelines. The SCC testing methods, which will be briefly re-introduced later in this chapter, were explained in detail in the course of these guidelines.

8.4 EXPERIMENTAL INVESTIGATION: RECYCLED SCC PRODUCTION

(a) General

A standard concrete mix, which will be considered as a reference mix, was designed to achieve 50 N/mm^2 at 28 days with a high slump of 60-180 mm and Vebe time of 0 - 3 s in accordance with the Building Research Establishment mix design method (BRE 1992). Mix proportions are shown in Table 8.1 and 8.2, while the complete mix design is shown in Appendix 2.

Table 8.1 Mix proportions of standard mixes for SCC (control mixes)

Material	Mass used (kg/m^3)	
	NA	RA
Cement	515	515
Water	205	205
Coarse aggregate	1015	990
Fine aggregate	625	610
Wet density	2360	2320

Table 8.2 Mix proportions of standard mixes for SCC with 30% PFA or RGD

Mix	Code	Mass (kg/m^3)					
		Cement	PFA	RGD	Water	Coarse aggregate	Fine aggregate
5	RAC-SCC-30PFA	410	175	0	185	970	600
6	RAC-SCC-30RGD	410	0	175	185	970	600

SCC is often produced when appropriate proportions and the correct dosages of the SP are used. The optimal dosage of SP can be determined by trial mixes. The SP enables the concrete to compact under its own weight and flow around reinforcement without causing any segregation or blockages. Too much SP in a concrete mix will cause segregation of coarse aggregates and an increase in setting time is also a possibility (Neville 1995).

(b) Testing methods

In this study, trial mixes were produced for each mix shown in Table 8.2 to identify the approximate amount of SP required for adequate workability of the mixture while maintaining its structure. Together with other concrete components, a trial mix will show if the amount of SP is able to meet the typical criteria set out for SCC either in the relevant standard, in general guidelines, or even those concluded from previous research and case studies when specific requirements are not yet standardised. Taking into account the requirements for successful SCC, the following testing methods were considered essential and therefore will be briefly described in the following paragraphs. More details of testing methods and SCC evaluation can be found in the original EFNARC guidelines; the major source of information supporting the following brief.

(i) Slump flow

The slump flow is used to ascertain the flowing ability of SCC with no obstructions. This test is based on the well known standard concrete slump test, with the same apparatus and gives an indication of the filling ability of the concrete and its resistance to segregation.

The base plate used in this test was made in the concrete laboratory of Edinburgh Napier University, in accordance with BS EN 12350-2: 2000 from a piece of plywood 900×900 mm as shown in Fig. 8.1 and given six coats of varnish to achieve a consistent surface finish, minimal absorbency, and minimum resistance to concrete flow. A cross was drawn centre ways in the board and perpendicular to both sides with two circles drawn on the centre of the board having a diameter of 200 mm for the central one and 500 mm for the outer. The inner circle's diameter is equal to the slump cone base diameter; the base should be exactly located on this circle when the slump of fresh concrete is measured.

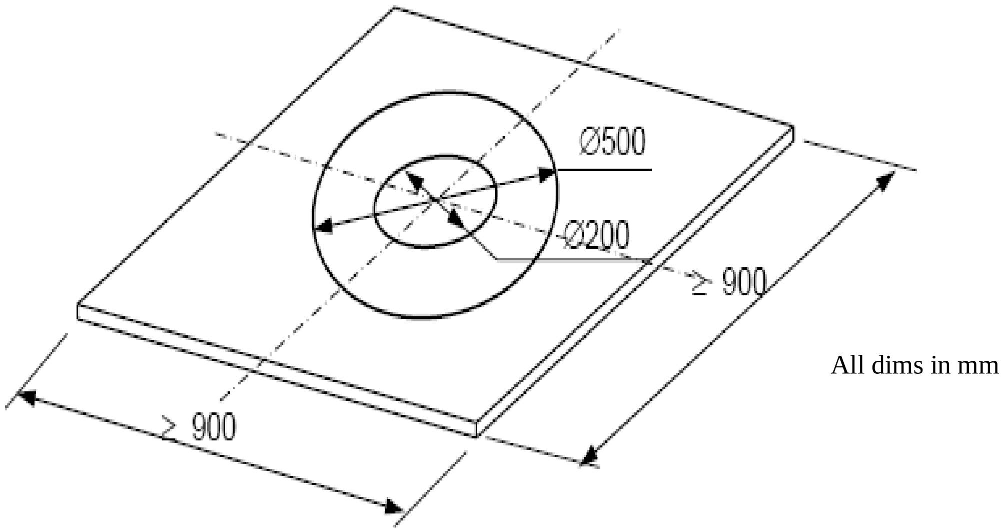


Fig. 8.1 Dimensions and marking of the base plate (EFNARC 2005)

Fig. 8.2 Slump cone located on the inner circle above the base plate

The cone and the board should be cleaned and dampened with a moist cloth before use, but care should be taken not to saturate the board or cone as this will affect the slump. The cone is placed over the small circle on the board with the funnel on top of it kept in position. Fresh concrete is gradually poured into the cone without tamping, and once filled the cone is lifted off upwards as soon as possible to avoid inaccurate results. When concrete spreading on the board is complete, the largest diameter of the flow spread and its diameter at right angles are averaged and recorded to the nearest 10 mm. This value is known as the slump flow of the concrete. According to EFNARC guidelines, three classes of slump flow are specified, class SF1 for a flow of 550 to 650 mm, SF2 for a flow of 660 to 750 mm and SF3 for a flow of 760 to 850 mm.

(ii) Spreading times of concrete (T_{500} and T_{final})

The concrete spreading times *i.e.* T_{500} and T_{final} are taken as indicative of the viscosity and flow rate of concrete. Slump flow and spreading times are carried out at the same time. The times are observed by means of a stopwatch accurate to ± 0.1 s during the slump flow test. The time the spreading concrete takes to reach the 500 mm diameter circle on the board is recorded as T_{500}, while the time taken to cease flowing on the board is recorded as T_{final}. In addition to the data recorded from the above tests, visual inspection of the concrete spread on the slump board is very important and should be taken into account. If the spreading circle is more than 500 mm, the concrete could be considered as meeting this requirement, but if it consists of a pile of aggregate in the centre of the board surrounded by a ring of cement slurry at the circumference it shows that the aggregate is not being transported uniformly throughout the mix; therefore segregation needs to be noted. In such situations, alterations will have to be made to the mix proportions before trials are continued. EFNARC guidelines specify two viscosity classes, class VS1 if $T_{500} \leq 2$ s and VS2 if $T_{500} > 2$ s.

(iii) L-box test

Dimensions and typical design of an L-box (EFNARC-2005) are shown in Fig. 8.3. The L-box used in this test was constructed in the concrete laboratory of Edinburgh Napier University, in accordance with BS EN 12350-10: 2007 and EFNARC-2005 as shown in Fig. 8.4. For a concrete to be considered as an SCC, it needs to pass the L-box test, in addition to a flow test. This test examines the passing ability of the concrete through normal reinforcement or congested reinforcement without causing any blockages or segregation of the aggregate. This is done by using two reinforcing bars (for normal) or three reinforcing bars (for congested) reinforcement at the base of the L-box's vertical hopper. The passing ability of the concrete (PA) is obtained from this test.

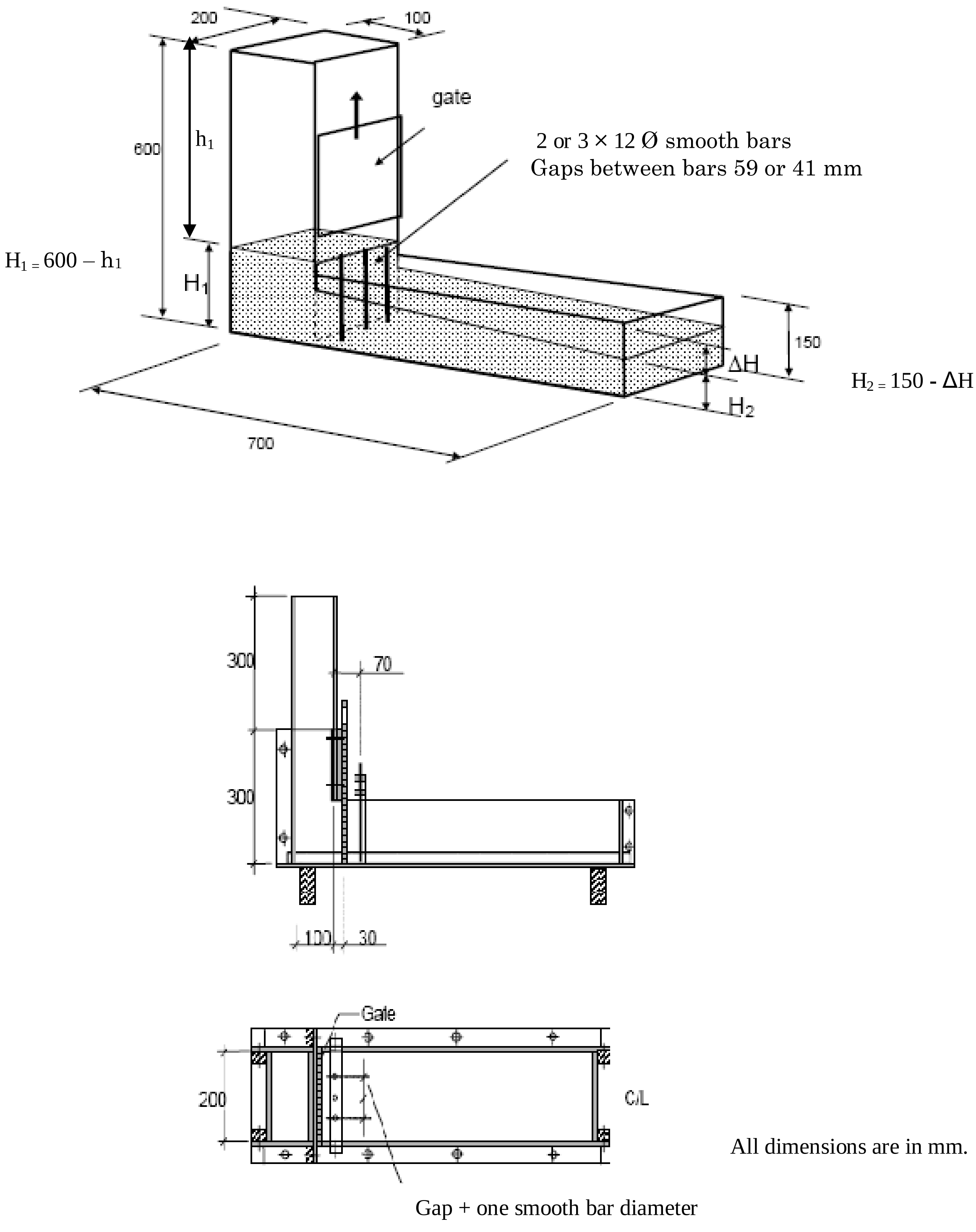


Fig. 8.3 Dimensions and typical design of L-box (EFNARC 2005)

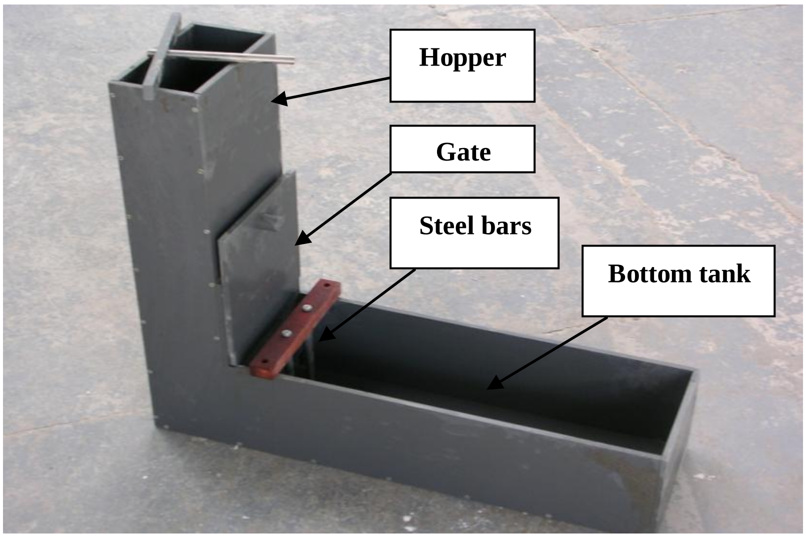


Fig. 8.4 L-box apparatus

The L-box shown in Fig. 8.4 was made of Perspex™ which provides a smooth surface for the concrete and is easy to clean and maintain. The dimensions of the vertical hopper are 600 mm high, 200 mm wide and 100 mm deep. For the purpose of this experiment the reinforcement used was two 12 mm diameter smooth bars spaced evenly across the section at a separation of 59 mm. The two bar system was used instead of three as it satisfied the general rule relating the gap and aggregate size; that is the spacing of reinforcing bars should be twice the diameter of the largest aggregate, which in this case was 20 mm. Three bar systems may be used for smaller aggregate.

Before conducting the test, the inner surfaces of the L-box were wiped with a damp cloth to remove any dust and ensure the gate opens easily. It is placed on a levelled surface and as close as possible to the concrete mixer; to reduce the distance the concrete has to be carried. While the gate is closed, fresh concrete is poured into the hopper and filled to the top. It is left to stand for 60 s. As the gate in the L-box is lifted upwards, the concrete will start to flow into the bottom section of the L-box as shown in Fig. 8.5.

Fig. 8.5 Concrete flow into the bottom tank of the L-box

Once the concrete stops flowing, the distance from the top of the L-box to the concrete inside the hopper is taken at different locations and the average of these values noted as h_1. Similarly, the distance between the concrete and the top of the L-box on the open side (the bottom tank) is measured at the edges and mid-point, averaged and noted as h_2. If there are any blockages behind the reinforcing bars as shown in Fig. 8.6, this should be reported.

Fig 8.6 Blockages behind reinforcing bars

The passing ability (PA) of the concrete is then calculated from Eqn 8.1:

$$PA = \frac{H_2}{H_1} \qquad \text{(Eqn 8.1)}$$

Where;

$H_1 = 600 - h_1$, and $H_2 = 150 - h_2$ (Dimensions are all in mm).

The closer the ratio PA is to one, the better the passing ability of concrete. According to EFNARC guidelines, two classes of passing ability are specified; PA_1 when $PA \geq 0.8$ with two reinforcing bars, and PA_2 when $PA \geq 0.8$ with three bars.

Fig. 8.6 demonstrates the passing ability (or lack thereof) of the aggregate through congested reinforcement. The mortar clearly segregates from the coarse aggregate due to extensive bridging between the small gaps. It also highlights the risks imposed by blockages in the shuttering at column-beam intersections. Without even distribution of all concrete components, the required strength will never be achieved and collapse of the structure becomes a serious possibility.

(iv) Trial mixes and fresh concrete results

It may be worth reminding the reader that the same materials which were tested for their fundamental properties in Chapter 4 of this study, and used to produce concrete mixes of Chapters 5, 6 and 7, were used in the current investigation. For the purposes of mix optimization, trial mixes were carried out to determine the appropriate dose of SP. In each mix, a quantity of SP (as a percentage of the cementitious material mass) was added starting with 0.6%; on the basis of experience gleaned from working with these materials. This was increased by 0.2% for each trial mix until the optimum amount was ascertained. This increment was chosen to be so small because it was observed (through preliminary trials) that the slump was very sensitive to SP content. The optimum amount was defined here as the quantity of SP that together with other concrete components has satisfied the SCC requirements. For the purpose of this investigation, the mixes listed in Table 8.3 were selected. Trial mixes were carried out on these following the procedure used in previous chapters; the exceptions are the mixing time which was increased to 6 minutes (Chopin 2004) and the PFA concrete samples' de-moulding time was more than 24 hours but with a maximum of 48 hours; to avoid breakage of samples.

Table 8.3 List of mixes selected to produce RAC-SCC with SP, PFA, and RGD

Mix	Code	Reason for selection
1	NAC-SCC-CM	Control mix of NAC
2	RAC-SCC-CM	Control mix of RAC
3	NAC-SCC-SP	To examine the effect of SP on the properties of NAC-SCC mix
4	RAC-SCC-SP	To examine the effect of SP on the properties of RAC-SCC mix and to obtain an indication about the possible strength compared to NAC-SCC concrete
5	RAC-SCC-SP-30PFA	To examine the effect of PFA on the properties of RAC-SCC mix and to obtain an indication about the possible strength compared to NAC-SCC and RAC-SCC-SP concrete
6	RAC-SCC-SP-30RGD	To examine the effect of RGD on the properties of RAC-SCC mix and to obtain an indication about the possible strength compared to NAC-SCC, RAC-SCC-SP, and RAC-SCC-SP-30PFA concrete

30% PFA and RGD were proven to be optimal (see Chapter 7)
The SP content will be determined by trial mixes

However, results of these mixes can be compared with their counterpart conventional concretes produced in Chapter 7. Preliminary trial mixes showed that the SCC control mixes were slightly dry, therefore water content of standard mix proportions listed in Table 8.1 has been adjusted as in Table 8.4 by adding 5% water as the first trials to produce standard slump failed; as a result the water content was adjusted to 215 kg/m^3 ($205 \times 1.05 = 215$) and w/c ratio becomes 0.42 instead of 0.40.

Table 8.4 Adjusted mix proportions of SCC concrete standard mixes

Mix	Code	Mass (kg/m^3)			
		Cement	Water	Coarse aggregate	Fine aggregate
1	NAC-SCC-CM (control mix)	515	215	1015	625
2	RAC-SCC-CM (control mix)	515	215	990	610

It may be more convenient to describe the trial process in general terms instead of describing, in a repetitive way, what had happened with each individual one, as most have failed a few trials carried out to satisfy the requirements of SCC in which trials were repeated several times. The first trials of all mixes incorporating an SP content of 0.6% of the cementitious materials have, as expected, failed to pass the slump flow test as is shown in Fig. 8.7. For those mixes having failed the slump flow test, the L-box test was obviously not needed as it would inevitably be a “fail”.

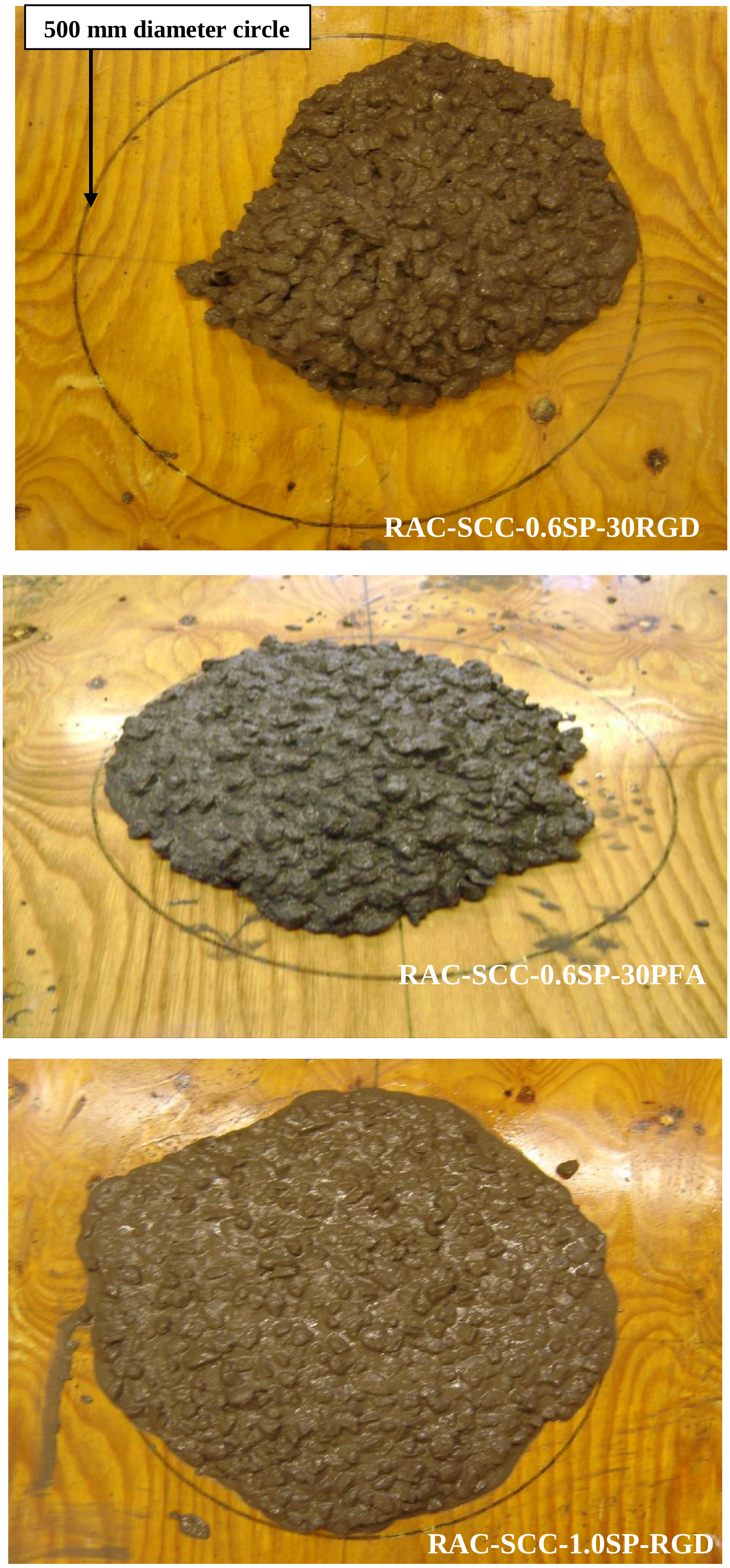


Fig. 8.7 Concrete failing the flow test

For a concrete to pass the slump flow test it must flow to an even spread around the slump board to a diameter of at least 500 mm (the large circle diameter), all the aggregate must be borne out to this limit without any segregation in the mix.

Once the first trial failed the flow test, the amount of SP was increased at a rate of 0.2% *per* trial until the mix successfully passes the flow test. In some cases, although the mix passed the flow test the concrete was not considered SCC; the reason is the observed segregation between coarse aggregate and cement paste as shown in Fig. 8.8. In such cases, the SP was considered to be greater than the optimum level; another trial with a reducing SP dosage at a rate of 0.1% *per* trial was repeated. The flow diameter is the average of the maximum spread diameter and the diameter perpendicular thereto as shown in Fig 8.9.

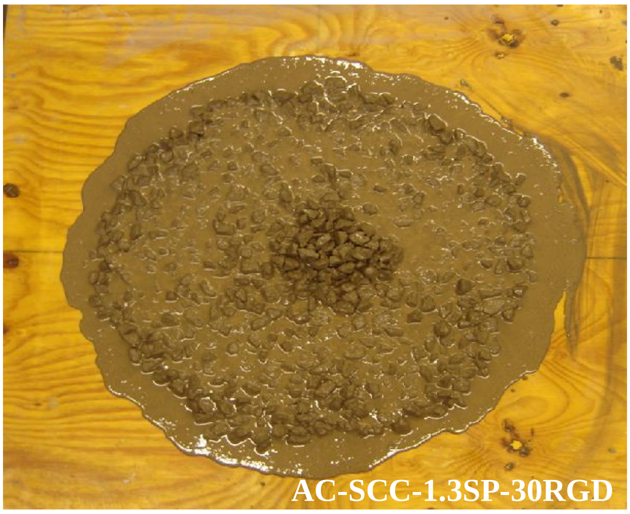


Fig. 8.8 Failed mixes due to segregation between coarse aggregate and cement paste although the mix passed the flow test

Fig. 8.9 Measuring flow diameter of a successful SCC mix

Fresh concrete mixes passing the flow table test were tested in the L-box (Fig. 8.10).

RAC-SCC-0.6SP

RAC-SCC-0.9SP

RAC-SCC-0.8SP-30RGD

Fig. 8.10 Concretes failing to complete an L-box test having passed the flow test

In a few cases, particularly at low SP contents, the mix had passed the slump flow test, but when it was put through the L-box test it failed as shown in Fig. 8.10.

Concrete trials which successfully completed the slump flow test by passing the 500 mm diameter spread without causing any segregation and also completing the L-box test by filling into the open tank of the L-box without causing any blockages around the reinforcement or segregation in the mix can be considered to be SCC. A sample is shown in Figs 8.11 and 8.12.

RAC-SCC-CM

RAC-SCC-1.1SP

RAC-SCC-1.0SP -30PFA

RAC-SCC-1.0SP -30RGD

Fig. 8.11 Samples of concrete successfully passing slump flow test

RAC-SCC-CM

RAC-SCC-1.0SP -30PFA

RAC-SCC-1.1SP-30RGD

Fig. 8.12 Samples of concrete successfully passing slump flow and L-box tests

However it should be noted that the trial mixes tested to satisfy SCC requirements were not straightforward compared to trials of conventional concrete. Trials have to be repeated many times, need careful adjustment, considerable labour and longer time. For accuracy, the testing was carried out by the same two people; the author and a laboratory technician. Results of the above experiments are summarised in Tables 8.5 and 8.6.

Table 8.5 The amounts of SP and WR found to meet SCC criteria for the given mix

Mix	Code	SP (%)	WR (%)
1	NAC-SCC-CM (control mix)	0	0
2	RAC-SCC-CM (control mix)	0	0
3	NAC-SCC-0.9SP	0.9	18
4	RAC-SCC-1.1SP	1.1	18
5	RAC-SCC-1.0SP -30PFA	1.0	18
6	RAC-SCC-1.1SP-30RGD	1.1	18

Results were in agreement with the results obtained in Chapter 4; Section 4.8 (e); Table 4.27; *e.g.* WR was 16% for 0.8% SP for a mix designed to achieve 10-30 mm slump. For the same material here WR was 18% for 0.9% SP for 60-180 mm slump.

Applying the findings in Table 8.4, the SCC proportions were adjusted accordingly as in Table 8.6.

Table 8.6 Mix proportions of SCC concrete (all mixes)

Mix	Code	Mass (kg/m^3)						
		Cement	Water		Coarse aggregate	Fine aggregate	PFA	RGD
			Original[a]	adjusted				
1	NAC-SCC-CM (control mix)	515	215	215	1015	625	0	0
2	RAC-SCC-CM (control mix)	515	215	215	990	610	0	0
3	NAC-SCC-0.9SP	515	215	177	1015	625	0	0
4	RAC-SCC-1.1SP	515	215	177	990	610	0	0
5	RAC-SCC-1.0SP-30PFA[b]	410	195	160	985	600	175	0
6	RAC-SCC-1.1SP-30RGD	410	195	160	985	600	0	175

[a]Original water is the mix design value
[b]Mixes 5 and 6 contains different cement content as it was designed to contain PFA and RGD

Table 8.7 show w/c and w/b ratios of SCC mixes.

Table 8.7 The w/c and w/b ratio of SCC mixes

Mix	Code	Cement (kg/m^3)	Binder (kg/m^3)	Original water (kg/m^3)	WR (%)	Adjusted water (kg/m^3)	w/c	w/b
1	NAC-SCC-CM (control mix)	515	515	215	0	215	0.42	0.42
2	RAC-SCC-CM (control mix)	515	515	215	0	215	0.42	0.42
3	NAC-SCC-0.9SP	515	515	215	18	177	0.34	0.34
4	RAC-SCC-1.1SP	515	515	215	18	177	0.34	0.34
5	RAC-SCC-1.0SP-30PFA	410	585	195	18	160	0.40	0.27
6	RAC-SCC-1.1SP-30RGD	410	585	195	18	160	0.40	0.27

Mix design of RAC-SCC with 30% PFA and RGD according to (BRE 1992) especially prepared for PFA concrete with slump of 60-180 mm and Vebe time 0-3 s is shown in Appendix 2. The workability as indicated by slump, concrete spread, and fluidity naturally improved as the SP content increased. Every mix has yielded different flow, T_{500} and T_{final}, fluidity, and passing ability when the SP was added at a particular level. Only data from successful mixes are reported here in Table 8.8.

Table 8.8 Workability of successful mixes of RAC-SCC

Mix	Code	w/c	w/b	Slump (mm)	Vebe (s)	Flow dia. (mm)	T_{500} (s)	T_{final} (s)	L-Box (PA)
1	NAC-SCC-CM	0.42	0.42	85	2	-	-	-	-
2	RAC-SCC-CM	0.42	0.42	70	4	-	-	-	-
3	NAC-SCC-0.9SP	0.34	0.34	285	-	710	2.18	15.2	0.89
4	RAC-SCC-1.1SP	0.34	0.34	275	-	690	2.34	13.5	0.85
5	RAC-SCC-1.0SP -30PFA	0.33	0.27	265	-	695	2.65	20.7	0.95
6	RAC-SCC-1.1SP-30RGD	0.33	0.27	270	-	710	1.58	10.5	0.95

Control mixes are standard vibrated mixes

Data in Table 8.8 showed that each mix is unique; each mix needed a certain amount of SP to satisfy SCC requirements. Concrete has to maintain its workability for a period of time to give operatives the chance to place it properly in the formwork. For successful mixes, the slump and slump flow were tested on the mix over the course of 90 minutes to see how the concrete would behave over time; loss of flow and slump were measured every 15 minutes. Slump and spread diameter measurements, to the nearest 5 mm, are given in Table 8.9.

Table 8.9 Slump and flow of SCC mixes at different times

Mix	Code	Average slump (mm) and flow diameter (mm) after a time of (min.):						
		5	15	30	45	60	75	90
3	NAC-SCC-0.9SP	285 (710)	270 (630)	260 (600)	235 (550)	200 (470)	80 (200)	40 (200)
4	RAC-SCC-1.1SP	275 (690)	220 (580)	210 (570)	190 (550)	180 (485)	110 (210)	50 (200)
5	RAC-SCC-1.0SP -30PFA	290 (780)	285 (755)	275 (710)	265 (675)	250 (640)	220 (575)	145 (430)
6	RAC-SCC-1.1SP-30RGD	260 (680)	245 (650)	220 (605)	190 (575)	170 (505)	95 (240)	45 (200)

The data in Table 8.9 are plotted in Figs 8.13 and 8.14.

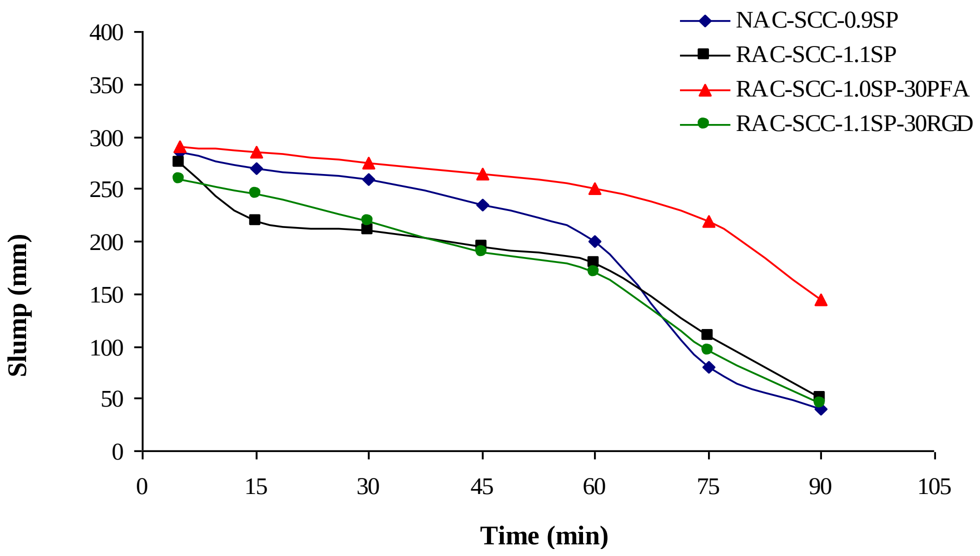


Fig. 8.13 Slump against time

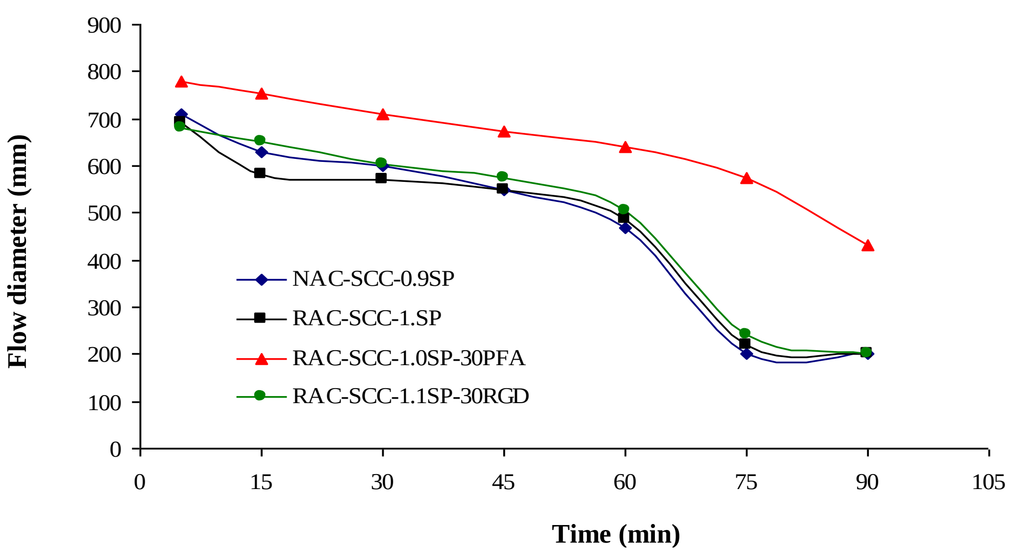


Fig. 8.14 Flow diameter against time

Figs 8.13 and 8.14 show that the recycled aggregate mix with PFA was superior compared to other mixes when slump loss and flow are concerned; steady decrease of both values were observed. Even after 90 minutes, a slump of 145 mm and flow diameter of 430 mm were measured; this however can be attributed to the high initial slump which was basically comes from the higher design slump of the standard mix (60 -180 mm), the influence of SP, the spherical grain shape on fluidity, and the lower porosity of PFA grains.

Data in Table 8.8 show that 34, 25, 12, and 23% of slump was lost after one hour for Mixes 3 to 6. This result indicates that the substitution of PFA and RGD to partially replace cement improves and maintains, and at the very least did not adversely influence, the workability of RAC-SCC. In particular, the RAC concrete mix without cement substitution (Mix 4) has remarkably showed steady slump values within the period of 15 to 50 minutes; a larger drop was noticed within the first 15 minutes when compared with the similar mix with natural aggregate (Mix 3), this however was expected due to the difference in absorption capacity. This feature was less pronounced in RAC-SCC mixes with PFA and RGD. A similar trend was observed for flow.

Red granite dust was shown to make good improvements to fresh properties of RAC-SCC concrete. Flow diameter of RGD mix (Mix 6) was better than PFA mix (Mix 5) and plain mix with SP (Mix 4); slump within 90 minutes was generally comparable to RAC-SCC without RGD. However, after 90 minutes all mixes, except the PFA mix, slumps were practically equal.

(v) Mechanical properties of RAC-SCC concrete

These experiments were undertaken to examine the possibility of producing superplasticized SCC using 100% RA, and to determine the possible strength ranges that can be achieved either with full cement content or with a 30% cement replacement by either fly ash or RGD. The two aggregates were not combined in these experiments as this would not give the true potential strength for a 100% RA mix *versus* a 100% NA mix.

Cube samples were cast and hardened concrete was tested in similar way to that outlined in previous chapters, the only exception was the SCC which was cast in cubes and beams without vibration. Table 8.10 presents the measured strengths at the indicated time.

Table 8.10 Strengths of the SCC mixes

Mix	Code	Compressive strength (N/mm^2) after:			Tensile strength (N/mm^2) after:	Flexural strength (N/mm^2) after:
		28 d	56 d	90 d	28 d	28 d
1	NAC-SCC-CM	76	79.4	82.4	3.93	7.32
2	RAC-SCC-CM	59.3	61.0	62.7	3.54	6.53
3	NAC-SCC-0.9SP	93	92.6	93.2	4.53	12.1
4	RAC-SCC-1.1SP	60.7	61.5	62.2	4.16	9.16
5	RAC-SCC-1.0SP -30PFA	57.6	57.3	59.6	2.97	6.42
6	RAC-SCC-1.1SP-30RGD	47.9	54.6	55.8	2.86	5.82

Results showed that the compressive strength of the SCC was increasing in a similar fashion to conventional concrete, regardless of aggregate type. The SP maintained workability, gave the required fluidity, contributed to form the slurry suspension needed to coat the aggregates and facilitate their relative movement. The SP also enabled the mix to be designed at low w/c ratio therefore producing better strengths. The lower w/c ratio's influence clearly appeared in SCC without cement substitution; the compressive, tensile and flexural strengths at 28 days of NAC-SCC-0.9SP (Mix 3) were relatively enhanced. The increased strengths were most likely due to the enhanced matrix of the concrete. Fracture surfaces of SCC samples are shown in Fig. 8.15.

NAC-SCC-CM RAC-SCC-CM

RAC-SCC-1.0SP-30PFA

NAC-SCC-0.9SP RAC-SCC-1.1SP-30RGD

Fig. 8.15 Fracture surface of SCC samples

Superplasticized RAC concrete without cement replacement did not benefit from enhanced matrix properties as comparable compressive strength was obtained at 28 days. However, the tensile and flexural strengths were marginally improved. The ultimate strength was most likely influenced by the strength of the recycled aggregate.

A reduction of strengths, particularly the tensile strength, was observed when 30% of the cement was replaced by PFA and RGD compared to all other mixes, the 90 day compressive strengths were less than the target mean strength (63 N/mm^2). The mix with PFA replacement was however better than its counterpart RGD mix. To make use of the observed benefit of PFA to the workability of SCC and perhaps to increase the strength, in addition to other well known advantages, PFA can be used as a supplement to cement instead of as a cement replacement.

8.5 SUMMARY

In general it can be said that it was possible to produce SCC with recycled aggregate. Strengths of RAC-SCC, though lower than similar NAC-SCC concrete, were acceptable and cover a wide range of strengths often required for small and medium size structures. In agreement with results obtained in Chapter 7 when RGD was used for ordinary RAC mixes, the use of RGD in RAC-SCC concrete is not recommended, particularly for medium to high strength mixes, as it leads to decreased strength. Enhanced performance was achieved when RGD was used with NAC-SCC; the successful large-scale utilisation of RGD in SCC would not only lower its cost, but also address the disposal and environmental problems of aggregate production.

CHAPTER 9

COMMENTS, CONCLUSIONS, AND RECOMMENDATIONS

9.1 GENERAL

One of today's major challenges facing the civil engineering community is to ensure modern society's standards and needs as expressed by the concept of sustainable development involving re-use and recycling of waste materials and industrial by-products, is cost-effective and truly eco-friendly. The conservation of primary materials has been the main aim of this thesis.

The quantities of C&DW and mineral by-products going to landfill have reached an unacceptable level in many places worldwide. Advances in demolition and processing technologies led to produce much higher quality recycled aggregates than those produced during the early days of recycling. The proportion of recycled aggregates has increased; each year about 250 to 270 million tonnes of aggregates are used; about 50 million tonnes accounting for 18.5-20% of this are supplied through recycling (British Geological Survey 2009). The aforesaid facts are the drivers behind this research.

In this study, chemical and mineral admixtures were successfully used to produce different RACs.

9.2 COMMENTS

From this study the following general comments can be made:

1. Natural aggregates are used needlessly for many applications; high-strength aggregates were used in secondary, ordinary, and structural concrete members requiring low- and medium-strength concrete for which secondary or recycled aggregate can be exploited without affecting concrete performance in either short- or long-terms.

2. The use of RA in new concrete is still very low; recycled materials from various sources now accounting for 17% of all aggregates used in the UK (British Geological Survey 2009). Concrete recyclers cited the difficulty of breaking up large concrete masses, the availability of NA at competitive prices in some places, project specifications, codes and standards, and the past low performance

of RAC as the major obstacles to use RA in construction. The key issue to increase the use of RAC in construction is to improve its performance. If recycled material use can be improved and promoted then it has the ability to put pressure on the construction industry for two key environmental concerns: landfill and quarrying.

3. From the standpoints of cost and availability of construction materials, the increased volume of concrete production and the expansion in the extraction of natural aggregates, the conservation of natural resources, the environmental implications of C&DW and other by-products, the improved quality of RA and the enhanced performance of RAC, these encouraging research results and the creation of new standards especially for RA, such unwarranted opposition to the widespread use of RAC can not be uphold. The construction industry must realise its responsibility to adopt more environmentally friendly approaches to construction.

4. If concrete works within any project are classified according to strength requirement, the type of aggregate can subsequently be specified for each category. Well processed recycled aggregates can cover a wide range of concrete applications including structural concrete. Concrete works implemented worldwide are able to accommodate RA when all concrete sections which need to achieve a compressive strength below 20 N/mm^2 are cast using RAC.

5. Due to relatively long experience with processing and production of recycled aggregate, the extra cost associated with processing of C&DW is lowered nowadays or even cut as the initial developing phase ends; when combined with governmental and social commitment to recycling, the increased cost of production and transportation of conventional natural aggregate, landfill tax, aggregate levy, and environmental concerns, the situation is expected to change gradually in favour of recycled aggregates.

6. The employment of effective portable crushing and processing equipments could play a significant role in keeping close control of the RA’s quality and making RAC more economical than NA. When such equipment becomes commercially available, not only the transportation cost can be avoided; other quality aspects can be controlled and the end product can be enhanced.

7. For most early research projects on recycled aggregates; and even in some recent ones, the recycled aggregates were produced from concrete cubes or masses crushed manually at 28 days in a laboratory. The use of RA produced from C&DW in recycling facilities is more suitable to assess the potential of recycling the C&DW in new concrete.

8. Fly ash is the most abundant mineral additive; it is inexpensive in many places worldwide. PFA is used in concrete to achieve several objectives; most are practical such as improving workability, lowering hydration heat, enhancing strength, and durability.

9.3 CONCLUSIONS

From the experimental laboratory-based work carried out in this study the following conclusions were drawn:

1. The data on aggregate recycling in new concrete and the use of admixtures was extensively reviewed, representing a state of the art report.

2. RAC exhibits similar behaviour to NAC, therefore concrete structures can be designed according to the prevailing theories used to design NAC members; however RAC is marginally deformable therefore attaining slightly more strain under similar stress (10 – 15 %)

3. In this study, a 100% RA, produced in an aggregate facility in Fife; Scotland, was used for coarse aggregate in all RAC mixes; the cube strength of all RAC concretes on average 33 % were below similar NAC at all ages, although they were comparable; this may be attributed to the superior quality of natural granite aggregate used to create NAC concrete mixes, and the presence of deleterious materials in RA. However, it proved possible, with the help of SP and mineral admixtures to produce good quality RAC concretes capable of satisfying the strength and stiffness requirements for concrete used in several applications, including structural concrete. In all cases examined in this study, the ultimate concrete strengths were influenced by aggregate type or more precisely the aggregate strength; the strength of RA limited the ultimate strength of RAC in exactly the same way as NA strength limits the ultimate strength of NAC. Improving the concrete matrix increases RAC strength, but eventually the maximum achievable strength is controlled by the weakest link in the concrete system: the matrix, the bond between the interfacial zone and the RA particles, and the RA grains themselves.

4. The amount of RA that may replace NA in RAC concrete sections should be linked to strength requirement; sections can be divided sensibly, for instance into low and medium strength for which RAC is used and higher strength sections for which NAC is used. However, the use of RA for a particular job should be evaluated on case-by-case basis.

5. The effect of blending RA and NA was a significant influence on the mechanical properties of concrete mixes tested. Replacement of 50% of the NA by RA has resulted in considerable improvement (37%) of the mechanical performance of superplasticized RAC.

6. The SP is a major component in producing good quality RAC incorporating mineral admixtures. The compatibility of an SP with the concrete components should be tested to avoid adverse reactions. Water reducer SPs ensure low w/c ratios and aid the workability. The author's calibration graphs are a good method to obtain the right amount of SP and the corresponding WR.

7. PFA could be used as a substitute for fine aggregate as it improves the quality of RAC concrete; a limit of 20-25% PFA fine aggregate replacement for superplasticized mixes is recommended. In this study, superplasticized RAC concrete with 25% fine aggregate replacement achieved compressive strengths of 56 N/mm^2 after 28 days and 66 N/mm^2 after 90 days, for a mix designed to attain 40 N/mm^2 after 28 days curing, although the cement content was reduced by 25%. Tensile strengths followed a similar trend; workability was enhanced. Such performance, in combination with the well known benefits of PFA to the long-term performance of concrete, makes this type of concrete more economical and satisfactory for many structural applications.

8. PFA is widely used to partially replace cement in ordinary NAC concrete without significantly affecting the concrete properties, particularly when combined with low w/c ratio and SP. The replacement is limited to 20-35% due to the requirement of additional water for wetting the finer fly ash which in turn increases w/c ratio and reduces the strength. In this study, 30% PFA replacement was used to create superplasticized RAC concretes (of a mix designed to achieve 50 N/mm^2 after 28 days). Strengths and stiffness of RAC were comparable to similar NAC concretes.

9. The commonly used equations for relating the mechanical properties of ordinary concrete cannot be applied for NAC and RAC produced with chemical or mineral admixtures. Design or quality control parameters should be obtained by direct measurement on representative samples. Pertinent

10. Considerable amounts of quarry wastes such as RGD powder are produced in aggregate quarries. In this study, the potential of utilising RGD in NAC and RAC both in conventional and SCC was investigated. RGD powder showed a similarity to PFA but was less reactive material. Conventional NAC concrete produced with RGD as a cement replacement at a level of 30% exhibited strength values either comparable or better than those of the reference mix. Strength values are marginally less than those of similar concrete made with PFA, but an equal or improved modulus of elasticity ensued. In the same way, RAC concrete with RGD showed strengths comparable to equivalent concrete with PFA, but with slightly larger (9%) elastic modulus. In contrast, the use of RGD for ordinary or SCC-RAC mixes is not recommended, particularly for medium to high strength mixes, as it leads to decreased strengths (37%).

11. Ordinary superplasticized NAC concrete mixes containing 30% RGD powder showed:

- **a.** Better mechanical properties compared with the control mix; the achieved strengths cover a wide range typically required in practice.
- **b.** Better early age strength than similar concrete with PFA.

c. Potential to cut the amount of cement, therefore providing cost saving, reducing pressure on natural resources, and restricting the risks inherent to aggregate production.

d. Good fresh properties.

e. Excellent surface finish.

12. It is possible to produce self-compacting concrete with recycled aggregate (SCC-RAC) and SP with or without mineral admixtures. Strengths of RAC-SCC, though lower than similar NAC-SCC concrete, were acceptable and cover a wide range of strengths often required for small and medium size structures.

9.4 RECOMMENDATIONS

1. To promote more recycling, concrete producers, and recyclers should be provided with tax credits as well as the removal of the unnecessary restrictions or barriers on the use of recycling materials. Crafting precise specifications taking into account sustainability requirements, educating structural engineers, designers, contractors, and the construction community on new construction materials and technologies, and enforcing contract plans during construction, are the keys to the wide utilisation of RAC concrete. Increase of dumping costs, particularly in developing countries could result in increased recycling activities. Public awareness of the resistance to utilisation and acceptance of RAC could be ended when contractors gain experience providing concrete that meets prescriptive specification requirements.

2. More future research RAC with and without mineral admixtures are recommended before a conclusive results can be drawn; limited research have been done on bond strength of RAC, abrasion resistance, deformation and modulus of elasticity, shear and flexural behaviour of plain and reinforced concrete section. Mixes could include 100% RA or blended aggregates *i.e.* RA and NA.

3. Studies on durability of RAC are also limited, particularly on those made with additives. This study or similar studies with blended aggregates could be extended to examine the durability aspects of RAC; Particularly permeability, carbonations, freezing and thawing resistance, creep and shrinkage. Studies may include both laboratory- and site-based large scale experimental testing.

4. Low strength RAC concrete incorporating limited amount of cement (or without cement) and crude material such as fly ash, RGD or GGBS designed to have high slump ordinary concrete with or without SP, or self compacting concrete is a subject of new point of research.

REFERENCES

A2E03 (2006) The American Committee on Properties of Concrete. [Internet] Available from: <http://onlinepubs.trb.org/onlinepubs/millennium/00022.pdf> [Accessed on 15 May 2007].

ACI (1987) Use of fly ash in concrete, Committee 226, ACI Materials Journal, 84(5), pp. 381-409.

ACI (1987) State of the art report on high strength concrete, Committee 363. High strength concrete. Seminar course manual- SCM 87-ACI.

Aggarwal P. & Gupta A. (2007) Effect of bottom ash as replacement of fine aggregate in concrete. Asian Journal of Civil Engineering, 8(1) pp. 49-62.

Agarwal S.K., Irshad M. & Malhotra S.K. (2000) Compatibility of superplasticizers with different cements. Construction and Building Materials, 14(5) pp. 253-259.

Aggregates Advisory Service (1999) Secondary and recycled aggregates uses in road construction under existing specifications. Aggregates Advisory Service. Digest 101. [Internet] Available from: < http://www.p2pays.org/ref/14/13992.pdf > [Accessed on 11 July 2008].

Aïtcin P.C. & Mehta P.K. (1990) Effect of coarse-aggregate characteristics on mechanical properties of high-strength concrete. ACI Materials Journal, 87(2) pp. 103-107.

Ajdukiewicz A. & Kliszczewicz A. (2002) Influence of recycled aggregates on mechanical properties of HS/HPC. Cement Concrete Composites, 24(2), pp. 269-279.

Akash R., Kumar J.H.A.N. & Misra S. (2007) Use of aggregates from recycled construction and demolition waste in concrete. Resources, Conservation and Recycling, 50(1) pp. 78-81.

Ala M., Khalid A. & Rajeh A. (2004) Prediction of the dynamic modulus of elasticity of concrete under different loading conditions. In: Proceedings of the International Symposium on Concrete Engineering and Technology, University of Malaya.

Alexander M.G. (1996) Aggregates and the deformation properties of concrete. ACI Materials Journal, 93(6) pp. 569-576.

Alexander M.G. & Milne T.I. (1995) Influence of cement blend and aggregate type on stress-strain behaviour and elastic modulus of concrete. ACI Materials Journal, 92(3) pp. 227-234.

American Coal Ash Association (2006) Coal Combustion Products (CCBs). [Internet] Available from: <http://www.rmajko.com/flyash.html> & <www. acaa.usa.org> [Accessed on 12 May 2007].

Amnon K. (2003) Properties of concrete made with recycled aggregate from partially hydrated old concrete. Cement and Concrete Research, 33(5) pp. 703-711.

Amnon K. (2004) Treatments for the improvement of recycled aggregate. Proc. Am. Soc. Civ. Engns, Journal of Materials in Civil Engineering, 16(6) pp. 597-603.

Andreas B. & Rühl M. (1989) The influence of recycled aggregate on the compressive strength and the elastic modulus of concrete. Darmstadt Concrete 13(PT) pp.131-141.

Angelakos D., Bentz E.C. & Collins M.P. (2001) Effect of concrete strength and minimum stirrups on shear strength of large members. ACI Structural Journal, 98(3), pp. 290-300.

Baalbaki W., Benmokrane B., Challal O. & Aïtcin P.C. (1991) Influence of coarse aggregate on elastic properties of high performance concrete. ACI Materials Journal 88(5) pp. 499-503.

Bairagi N.K. & Kishore R. (1993) Behaviour of concrete with different proportions of natural and recycled aggregates. Resource Conservation and Recycling, 9(3) pp. 109-126.

Banfill P.F.G. (1982) An experimental study of the effect of pfa on the rheology of fresh concrete and cement paste. In: Proceedings of the International Symposium on the Use of pfa in Concrete, Eds. J.G. Cabrera & A.R. Cusens, University of Leeds, Leeds, UK. pp. 161-171.

Bashar T. & Ghassan N. (2008) Properties of concrete contains mixed colour waste recycled glass as sand cement replacement. Construction and Building Materials, 22(5) pp. 713-720.

BERR (2008) Technical note. [Internet] Available from: <http://www.berr.gov.uk/aboutus/corporate/performance/performance_Framework/page14309.html> [Accessed on 10 November 2008].

Beshr H., Almusallam A.A. & Maslehuddin M. (2003) Effect of coarse aggregate quality on the mechanical properties of high strength concrete. Construction and Building Materials, 17(2) pp. 97-103.

Bharat Book Bureau (2009) World cement forecasts for 2012-2017. [Internet] Available from: <http://www.bharatbook.com/World-Cement-Forecasts-for-2012-2017.asp> [Accessed on 10 January 2009].

Bonen D. & Shah S.P. (2005) Fresh and hardened properties of self-consolidating concrete. Progress in Structural Engineering Materials Journal, 7(1) pp. 14-26.

Bouzoubaâ N. & Fournier B. (2003) Optimization of fly ash content in concrete. Part I: Non-air-entrained concrete made without superplasticizer, Cement and Concrete Research, 33(7), pp. 1029-1037.

BRE (1992) Design of normal concrete mixes. Building Research Establishment. Department of the Environment, UK.

BRE (2009) Concrete industry supports responsible sourcing. Constructing the Future, 39, p. 3.

British Geological Survey (2009) Sustainable utilization of quarry by-products. [Internet] Available from: <http://www.sustainableaggregates.com/docs/revs/t2c_suqbp.pdf> [Accessed on 25 November 2008].

British Standards Institution BS 6543: 1985. Use of industrial by-products and waste materials in building and civil engineering. BSI. London.

Buck A.D. (1977) Recycled concrete as a source of aggregate. ACI Materials Journal, 74(22) pp. 212-219.

Bui D.D., Hu J. & Stroeven P. (2005) Particle size effect on the strength of rice husk ash blended gap-graded Portland cement concrete. Cement and Concrete Composites, 27(3) pp. 357-366.

Bui V.K. & Shah S.P. (2002) Rapid methods for testing quality of fresh self-consolidating concrete. In: Proceedings of the 1st North American Conference on the Design and Use of Self-compacting Concrete. 12-13 November, 2002, Chicago, USA, Centre for Advanced Cement Based Materials at Northwestern University, pp. 281-285.

Burak F. (2007) Utilisation of high volumes of limestone quarry waste in concrete industry: self-compacting concrete case. Resources, Conservation and Recycling, 51(4) pp. 770-791.

Campbell S.A. (2008) An experimental analysis of the behaviour of polymeric and ceramic pipe-wall materials subjected to the action of a cavitating water jet. PhD thesis, Edinburgh Napier University.

Carette G.G., Bilodeau A., Chevrier R.L. & Malhotra V.M. (1993) Mechanical properties of concrete incorporating high volumes of fly ash from sources in the U.S. ACI Materials Journal, 90(6) pp. 535–544.

Carpenter A. (1994) Stone circle. Building, 36(9) pp. 54-55.

Cengiz D.A (2003) Accelerated carbonation and testing of concrete made with fly ash. Construction and Building Materials, 17(3) pp. 147-152.

Cengiz D.A. & Cahit B. (2007) Wet and dry cured compressive strength of concrete containing ground granulated blast-furnace slag. Building and Environment, 42(7) pp. 3060-3065.

Chandra S. & Björnström J. (2002) Influence of superplasticizer type and dosage on the slump loss of Portland cement mortars - Part II. Cement and Concrete Research, 32(10), pp. 1613-1619.

Chevin D. (1997) Money for old rubble. In: Proceedings of the CSS Conference on Sustainable Road Maintenance. Specifications for Recycled Aggregates in Roads. 18 November, Leamington Spa, UK, Building, 3, pp. 70-71.

Chopin D., Larrard D.L. & Cazacliu B. (2004) Why do HPC and SCC require a longer mixing time? Cement and Concrete Research, 34(12) pp. 2237-2243.

CIRIA (1993) Construction Industry Research and Information Association, Environmental issues in construction: a review of issues and initiatives relevant to the building, construction and related industries, CIRIA, London.
Collins R.J. (1993) Reuse of demolition materials in relation to specifications in the UK. Demolition and reuse of concrete and masonry: guidelines for demolition and reuse of concrete and masonry. In: Proceedings of the third international RILEM symposium on demolition and reuse of concrete masonry, 24–27 October, Odense, Denmark, E&FN Spon, pp. 49–56.

Collepardi M. (1998) Admixtures used to enhance placing characteristics of concrete. Cement Concrete Composites, 20(2-3) pp. 103-112.

Crawford T. (2007) A study by Tim Crawford, Group Economist, Halifax Bank of Scotland, now Lloyds Banking Group. [Internet] Available from: <https://www.tiscali.co.uk/news/newswire.php/property/themovechannel/news/2007/09/3/uk-reigns-supreme-in-europe.html> [Accessed on 10 October 2007].

Dae J.M. & Han Y.M. (2002) Effect of pore size distribution on the qualities of recycled aggregate concrete KSCE Journal of Civil Engineering, 6(3) p. 289-295.

Davies D.R. & Kitchener J.N. (1996) Massive use of pulverised fuel ash in concrete for the construction of a U.K. power station. Waste Management, 16(1-3) pp. 169-180.

DEFRA (2008) Waste Strategy for England 2007. Internet] Available from: <http://www.defra.gov.uk/environment/waste/strategy/strategy07/pdf/waste07-summary.pdf> [Accessed on 10 November 2008].

DeVenny A. (1999) Recycling of demolished masonry rubble. PhD thesis, Napier University, Edinburgh.

DeVries P. (1996) Concrete recycled: crushed concrete aggregate. In: Proceedings of the International Conference: Concrete in the Service of Mankind. I, Concrete for Environmental Enhancement and Protection, 24-28 June, Dundee, Scotland, pp. 121-131.

Dhir R.K, Zhu W.Z. & McCarthy M.J. (1998) Use of Portland pfa cement in combination with superplasticizer admixtures. Cement and Concrete Research, 28(9) pp. 1209-1216.

Dhir R.K. (1998) Recycled concrete aggregate for use in BS 5328 designated mixes. Concrete Technology Unit, University of Dundee, report CTU/498.

Dhir R.K., McCarthy M.J. & Tittle P.A.J. (2000) Use of conditioned PFA as a fine aggregate component in concrete. Materials and Structures, 33(1), pp. 38-42.

Dhir R.K. (2000) Recycled aggregate concrete (RAC): properties of aggregate and beams made from RAC. In: Proceedings of the International Conference: Concrete in the Service of Mankind. I, Concrete for Environmental Enhancement and Protection, 24-28 June, Dundee, Scotland, pp. 141-149.

Dhir R.K. (2001) Resolving application issues with the use of recycled concrete aggregate. University of Dundee, Concrete Technology Unit, report CTU/1601.

Dhir R.K. & Paine K.A. (2003) Demonstration project utilizing coarse recycled aggregates. Concrete Technology Unit, University of Dundee, report CTU/2403.
Dinakar P., Babu K.G. & Manu S. (2008) Durability properties of high volume fly ash self compacting concretes. Cement and Concrete Composites, 30(10) pp. 880-886.

Domone P.J. & Jin P. (1999) Properties of mortar for self-compacting concrete. In: Proceedings of 1st International RILEM Symposium on SCC. Stockholm, Sweden: RILEM publications, SARL, pp. 109-120.

Domone P.L. (2006) Self-compacting concrete: an analysis of 11 years of case studies. Cement and Concrete Composites, 28(2) pp. 197-208.

ECOBA (2008) The European Coal Combustion Products Association. Statistics for 2004. [Internet] Available from: <http://www.ecoba.com/news,document.html?id=68> [Accessed on 25 August 2008].

ECOBA (2009) Fly ash utilization in concrete. [Internet] Available from: <http://www.ecoba.com/ecobaccpexs.html> [Accessed on 22 January 2009].

EFNARC (2002) Specification and Guidelines for Self-Compacting Concrete. [Internet] Available from: < http://www.efnarc.org/pdf/SandGforSCC.PDF> [Accessed on 16 June 2008].

EFNARC (2005) Specification and Guidelines for Self-Compacting Concrete. [Internet] Available from: < http://www.zementindustrie.at/file_upl/sccguidelines_may2005.pdf > [Accessed on 16 June 2008].

Etxeberria M, Vázquez E., Marí A. & Barra M. (2007) Influence of amount of recycled coarse aggregates and production process on properties of recycled aggregate concrete. Cement and Concrete Research, 37(5), pp. 735-742.

European Commission Report (1999) Construction and demolition waste management practices and their economic impacts. [Internet] Available from: <http://europa.eu.intcomm/environmentwastestudies/cdwc&dw report.htm> [Accessed on 20 May 2007].

Felekoglu B. & Baradan B. (2003) Utilisation of limestone powder in self-levelling binders. In: Proceedings of the International Symposium on Advances in Waste Management and Recycling. Dundee, UK. Thomas Telford Publishing, pp. 475-484.

Felekoglu B., Kamile T., Bülent B., Akin A. & Bahadir U. (2006) Effect of fly ash and limestone fillers on the viscosity and compressive strength of self-compacting repair mortars. Cement and Concrete Research, 36(9) pp. 1719-1726.

Ferraris C., Brower L., Ozyildirim C. & Daczko J. (2000) Workability of self-compacting concrete. In: Proceedings of the International Symposium on High-performance Concrete, 25-27 September, Florida, USA: National Institute of Standards and Technology, pp. 398-407.

Fong W.F.K. & Jaime S.K.Y. (2002) Production and application of recycled aggregates. In: Proceedings of International Conference on Innovation and Sustainable Development of Civil Engineering in the 21st century, 1-3 August, Beijing, China. [Internet] Available from:

<http://www.cedd.gov.hk/tc/services/recycling/doc/prod_appl_ra.pdf>. [Accessed on 8 May 2008].

Frondistou Y.S.T. (1977) Waste concrete as aggregate for new concrete. ACI Materials Journal, 74(8) pp. 373-376.

Ganesh B. K. & Rao G.S.N. (1993) Efficiency of fly ash in concrete. Cement and Concrete Composites, 15(4), pp. 223-229.

Ganesh B.K & Siva N.R. (1996) Efficiency of fly ash in concrete. Cement and Concrete Research, 26(3) pp. 465-474.

Gao P.-W., Lu X.-l, Lin H., Li X. & Hou J. (2007) Effects of fly ash on the properties of environmentally friendly dam concrete. Fuel, 86(7-8), pp. 1208-1211.

Gerardu J.J.A. & Hendriks C.F. (1985) Recycling of road pavement material in The Netherlands. Rijkswaterstaat Communication, 38, The Hague.

Ghafoori N. & Sukandar B.M. (1995) Abrasion resistance of concrete blocks pavers. ACI Materials Journal, 92(1), pp. 25-37.

Giaccio G., Rocco C., Violini D., Zappitelli J. & Zerbino R. (1992) HSC incorporating different coarse aggregates. ACI Materials Journal, 89(3) pp. 242-246.

Gilpin R., Menzie W. & David H.H. (2004) Recycling of construction debris as aggregate in the mid-Atlantic region, USA. Resources, Construction and Recycling, 42(3) pp. 275-294.

Gonzalez B.F. & Martinez A. (2008) Concretes with aggregates from demolishing waste and silica fume: materials and mechanical properties. Building and Environment, 43, pp. 429-437.

Guangcheng L., Xinyou W. & Youjun X. (2002) Very high performance concrete with ultrafine powders. Cement and Concrete Research, 32(4) pp. 601-605.

Hansen T.C. & Narud H. (1983) Strength of recycled concrete made from crushed concrete coarse aggregate. Concrete International, 5(1), pp. 79-83.

Hansen T.C. (1985) Elasticity and drying shrinkage of recycled aggregate concrete. ACI Materials Journal, 82(5) pp. 82-52.

Hansen T.C. & Narud H. (1992) Recycling of Demolished Concrete and Masonry, RILEM, London, E & FN Spon.

Hansen T.C. (1996) Recycled aggregate and recycled aggregate concrete. Second state-of-the-art report developments 1945-1985. Materials and Structures, 111, RILEM.

Harun T. (2009) Fuzzy logic model for the prediction of bond strength of high-strength lightweight concrete. Advances in Engineering Software, 40(3) pp. 161-169.

Hasaba S., Kawamura M., Torik K. & Takemoto K. (1981) Drying shrinkage and durability of concrete made of recycled concrete aggregates. Transactions of the Japan Concrete Institute, 3, pp. 55-60.

Hassan K.E., Cabrera J.G. & Maliehe R.S. (2000) The effect of mineral admixtures of the properties of high-performance concrete. Cement and Concrete Composites, 22(4), pp. 267-271.

Henry H.C.W. & Albert K.H.K. (2008) Packing density: a key concept for mix design of high performance concrete. [Internet] Available from: <http://www.hkpc.org/hkiemat/mastec05_program_notes/Prof.%20Albert%20KWAN.pdf> [Accessed on 14 March 2008].

Higuchi M. (1998) State of the art report on manufacturing of self-compacting concrete. In: Proceedings of the International Workshop on Self-Compacting Concrete, Kochi, Japan, pp. 360-367.

Ho D.W.S., Sheinn A.M.M., Ng C.C. & Tam C.T. (2002) The use of quarry dust for SCC applications. Cement and Concrete Research, 32(4) pp. 505-511.

Horszczaruk E. (2008) Mathematical model of abrasive wear of high-performance concrete. Wear, 264(2) pp. 113-118.

How J.C., Tsong Y. & Chen K.H. (2003) Use of building rubbles as recycled aggregates. Cement and Concrete Research, 33(1) pp. 125-132.

Hughes D.G. (2002) Self-compacting concrete: case studies show benefits to precast concrete producers. In: Proceedings of the 1st North American Conference on the Design and Use of Self-compacting Concrete. 12-13 November, 2002, Chicago, USA, Centre for Advanced Cement Based Materials at Northwestern University, pp. 405-412.

Ibrahim S., Shahram V., Ahmed E. & Mohammed F. (1996) The use of solid waste materials as fine aggregate substitutes in cementitious concrete composites. In: Proceedings of Semisesquicentennial Transportation Conference, Iowa State University, USA. [Internet] Available from: <http://www.ctre.iastate.edu/pubs/semisesq/session2/shehata/index.htm > [Accessed on 5 June 2008].

Illston J.M. & Domone P.L.J. (2001) Construction Materials: Their Nature and Behaviour, 3rd edition, London, Spon.

Isaia A.L.G., Gastald I. & Moraes R. (2003) Physical and pozzolanic action of mineral additions on the mechanical strength of high-performance concrete. Cement and Concrete Composites, 25(1) pp. 69-76.

Jackson N. & Dhir K.R. (1996) Civil Engineering Materials, New York, Palgrave Macmillan.

Jane G.P., Dal M.D.C. & Kazmierczak C.S. (2004) Study of the modulus of elasticity of polymer concrete compounds and comparative assessment of polymer concrete and Portland cement concrete. Cement and Concrete Research, 34(11) pp. 2091-2095.

Jianzhuang X., Jiabin L. & Zhang C. (2005) Mechanical properties of recycled aggregate concrete under uniaxial loading. Cement and Concrete Research, 35(6) pp. 1187-1194.

Josette C., Michael S., & Franco E.M. (2006) Utilization of pulverized fuel ash in Malta. Waste Mangement, 26(8) pp. 853-860.

Kakizaki M., Harada M., Soshiroda T., Kubota S., Ikeda T., Kasai Y. (1988) Strength and elastic modulus of recycled aggregate concrete. In: Proceedings of 2nd International RILEM Symposium on Demolition and Reuse of Concrete and Masonry. Tokyo, Japan, pp. 565-574.

Katharina K. (1997) Bearing and deformation of concrete with recycled aggregate, Darmstadt Concrete, 97, pp. 97-105.

Kayali O. (2008) Fly ash lightweight aggregates in high performance concrete. Construction and Building Materials, 22(12) pp. 2393-2399.

Ke-Ru W., Bing C., Yao W. & Zhang D. (2001) Effect of coarse aggregate type on mechanical properties of high-performance concrete. Cement and Concrete Research, 31(10) pp. 1421-1425.

King B. (2005) Making better concrete, guidelines to using fly ash for higher quality, eco-friendly structures. California, Green Building Press.

Kiyoshi E., Kohji T., Akira N., Hitoshi K., Kimihiko S. & Masafumi N. (2007) Application of recycled coarse aggregate by mixture to concrete construction. Construction and Building Materials, 21(7) pp. 1542-1551.

Khalaf F.M. & DeVenny A.S. (2002) New tests for porosity and water absorption of fired clay bricks. Journal of Materials in Civil Engineering, ASCE, 14(4) pp. 334-337.

Khalaf F.M. & DeVenny A.S. (2004) Recycling of demolished masonry rubble as coarse aggregate in concrete: a review. Journal of Materials in Civil Engineering, 16(4) pp. 331-340.

Khaldoun R. (2007) Mechanical properties of concrete with recycled coarse aggregate. Building and Environment, 42(1), pp. 407-415.

Khayat K.H. (1999) Workability, testing and performance of self-consolidating concrete. ACI Materials Journal, 96(3) pp. 346-353.

Khayat K.H., Hu C. & Monty H. (1999) Stability of SCC, advantages and potential applications. In: Proceedings of 1st International RILEM Symposium on SCC. Stockholm, Sweden: RILEM publications, SARL, pp. 554-459.

Kumar V. & Prasad M. M. (2003) Effect of fly ash and silica fume on the static modulus of elasticity of a concrete mix. Role of Cement Science in Sustainable Development, pp. 203-211.

Kwan A.K.H. & Mora C.F. (2001) Effects of various shape parameters on packing of aggregate particles. Magazine of Concrete Research, 53(2) pp. 91-100

Kyoto (2007) Kyoto Protocol to the United Nations Framework Convention on Climate Change. [Internet] Available from:
< http://unfccc.int/resource/docs/convkp/kpeng.html > [Accessed on 12 May 2007].

Landfill Tax (1996) Statutory Instrument 1996 No. 1527. [Internet] Available from: < http://unfccc.int/resource/docs/convkp/kpeng.html > [Accessed on 12 March 2006].

Levy S.M. (2004) Durability of recycled aggregate concrete: a safe way to sustainable development. Cement and Concrete Research, 34(11) pp. 1975-1980.

Libyan information Organisation (2008) Infrastructures in Libya: an overview, Report No 310-2008 issued by General Council Planning of Libya (available in Arabic).

Liu B., Xie Y., Zhou S. & Yuan Q. (2000) Influence of ultrafine fly ash composite on the fluidity and compressive strength of concrete. Cement and Concrete Research, 30(9) pp. 1489-1493.

MacNeil J. (1993) Wasted opportunities. Building, 12, pp. 47-49.

McCarthy M.J. & Dhir R.K. (1999) Towards maximizing the use of fly ash as a binder. Fuel, 78(2), pp. 121-132.

McCarthy M.J. & Dhir P.K. (2005) Development of high volume fly ash cements for use in concrete construction. Fuel, 84(11) pp. 1423-1432.

McLaughlin J. (1993) A review of the prospects for the greater use of recycled and secondary aggregates in construction. Concrete, 27(6) pp. 16-18.

McNulty A. & Khalaf F.M. (2005) Recycling demolished building rubble as aggregate in reinforced concrete beams. Unpublished BEng dissertation. School of the Built Environment, Napier University, Edinburgh.

Malhotra V.M. (1989) Structural concrete incorporating high volume of ASTM class F fly ash. ACI Materials Journal, 86(5) pp. 507-514.

Malhotra V.M. (1997) Innovative applications of supperplasticizers in concrete: a review. In: Proceedings of the 5th CANMET/ACI International Conference on Superplasticizers and other Chemical Admixtures, October, Rome, Italy, ACI special publication SP-173, pp. 271-314.

Mangaraj B.K. & Krishnamoorthy S. (1994) Use of pond fly ash as part replacement for mortar and concrete. Indian Concrete Journal, 5, pp. 279-282.

Mansur M.A., Wee T.H. & Cheran L.S. (1999) Crushed bricks as coarse aggregate for concrete. ACI Materials Journal, 96(4) pp. 478-484.

Maslehuddin M., Abdulaziz I., Mana A., Mohammed S. & Huseyin S. (1989) Effect of sand replacement on the early age strength gain and long term corrosion resisting characteristic of fly ash concrete. ACI Materials Journal, 86(1) pp. 58-62.

Massazza F. (1993) Pozzolanic cements. Cement and Concrete Composites, 15(4), pp. 185-214.

Mazloom M., Ramezanianpour A.A. & Brooks J.J. (2004) Effect of silica fume on mechanical properties of high-strength concrete. Cement and Concrete Composites, 26(4), pp. 347-357.

Mehta P.K. (1985) Influence of fly ash characteristics on the strength of Portland-fly ash mixture. Cement and Concrete Research, 15(4), pp. 669-674.

Mehta K.R. (1986) Properties of fresh concrete containing large amounts of fly ash. Cement and Concrete Research, 16(2), pp. 227-238.

Mehta K.R. (1988) Compressive strength of low cement/high fly ash concrete. Cement and Concrete Research, 18(1) pp. 571-583.

Mehta P.K. & Aïtcin P.C. (1990) Principles underlying production of high performance concrete. Cement Concrete Aggregate, 12(2) pp. 70-78.

Mehta P.K. (2002) High-performance high-volume fly ash concrete for sustainable development. International Workshop on Sustainable Development and Concrete Technology, Concrete International, 24(7) pp. 30-34. [Internet] Available from: <http://www.ctre.iastate.edu/pubs/sustainable/mehtasustainable.pdf > [Accessed on 20 February 2006].

Memon A.H., Radin S.S, Zain M.F.M. & Trottier J.-F. (2002) Effect of mineral and chemical admixture on high strength concrete in sea water. Cement and Concrete Research, 32(3), pp. 373-377.

Mouli M. & Khelafi H. (2008) Performance characteristics of lightweight aggregate concrete containing natural pozzolan, Building and Environment, 43(1), pp. 31-36.

Mulheron M. (1988) The recycling of demolition debris: current practice, products and standards in the United Kingdom. In: Proceedings of the Second International Symposium on Demolition and Reuse of Concrete and Masonry, Tokyo, Japan, pp. 510-519.

Mustafa Ş., Heru A.C. & İsmail Ö.Y. (2006) The effect of chemical admixtures and mineral additives on the properties of self-compacting mortars. Cement and Concrete Research, 28(5) pp. 432-440.

Naik T.R., Sivasundaram V. & Singh S.S. (1991) Use of high-volume class F fly ash for structural grade concrete. [Internet] Available from: < http://www.uwm.edu/Dept/CBU/Papers/1991%20CBU%20Reports/CBU%201991-04.pdf > [Accessed on 15 April 2009].

Naik T.R., Singh S.S., & Hossain M.M. (1994) Abrasion resistance of concrete as influenced by inclusion of fly ash. Cement and Concrete Research, 24(2), pp. 303-312.

Neville A.M. (2003) Neville on concrete. Farmington Hills, Michigan, ACI International.

Neville A.M. & Brooks J.J. (1990) Concrete technology. New York, Longman.

Neville A.M. (1995) Properties of concrete. 4th edition, London, Longman Scientific & Technical Ltd.

Nik D.O. (2005) Recycled aggregate concrete. Cement and Concrete Composites, 27(2), pp. 315-318.

Niro G.D., Dolara E. & Cairns R. (1998) Properties of hardened RAC for structural purposes. In: Proceedings of Conference on Sustainable Construction, Dundee, Scotland, UK, pp. 177-187.

Noguchi S.G. & Tomosawa F. (1999) Rheological approach to passing ability between reinforcing bars of self-compacting concrete. In: Proceedings of 1st International RILEM Symposium on SCC. Stockholm, Sweden: RILEM publications, SARL, pp. 59-70.

Nuno A., Branco J. & José R.S. (2007) High-performance concrete with recycled stone slurry. Cement and Concrete Research, 37(2) pp. 210-220.

Okamura H. & Ouchi M. (1999) Self-compacting concrete: development, present use and future. In: Proceedings of 1st International RILEM Symposium on SCC. Stockholm, Sweden: RILEM publications, SARL, pp. 3-14.

O'Mahony M. (1990) Recycling of materials in civil engineering. DPhil thesis, University of Oxford.

Omoto T. & Ozawa K. (1999) Recommendation for self-consolidating concrete. Japan Society of Civil Engineers, JSCE Concrete Engineering Series, 31, p. 77.

Oner A., Akyuz S. & Yildiz R. (2005) An experimental study on strength development of concrete containing fly ash and optimum usage of fly ash in concrete. Cement and Concrete Research, 35(6) pp. 1165-1171.

Oscar E.M. (1999) Coal fly ash: a retrospective and future look. Fuel, 78(2) pp. 133-136.

Owsiak Z. (2004) Alkali-aggregate reaction in concrete containing high-alkali cement and granite aggregate. Cement and Concrete Research, 34(1) pp. 7-11.

Papayianni I., Tsohos G., Oikonomou N. & Mavria P. (2005) Influence of superplasticizer type and mix design parameters on the performance of them in concrete mixtures. Cement and Concrete Composites, 27(2), pp. 217-222.

Poon C.S.L. & Wong Y.L. (2000) A study on high strength concrete prepared with large volumes of low calcium fly ash. Cement and Concrete Research, 30(3) pp. 447-455.

Poon C.S.L. (2004) Influence of moisture state of natural and recycled aggregates on the slump and compressive strength of concrete. Cement and Concrete Research 34(1), pp. 31-36.

Qin W. (2004) What role could concrete technology play for sustainability in China? In: Proceedings of the International Workshop on Sustainable Development and Concrete Technology. 20–21 May, Beijing China. Iowa State University. [Internet] Available from: <http://www.ctre.iastate.edu/pubs/sustainable/qinrole.pdf> [Accessed on 2 February 2007].

Rafat S. (2003a) Effect of fine aggregate replacement with class F fly ash on the mechanical properties of concrete. Cement and Concrete Research, 33(4) pp. 539-547.

Rafat S. (2003b) Effect of fine aggregate replacement with class F fly ash on the abrasion resistance of concrete. Cement and Concrete Research, 33(11) pp. 1877-1881.

Rafat S. (2004) Performance characteristics of high-volume class F fly ash concrete. Cement and Concrete Research, 34(3) pp. 487-493.

Rajamane N.P., Annie J.P. & Ambily P.S. (2007) Prediction of compressive strength of concrete with fly ash as sand replacement material. Cement and Concrete Composites, 29(3) pp. 218-223.

Ramazan D., Türkmen İ. & Mehmet B.K. (2004) Relationship between ultrasonic velocity and compressive strength for high-volume mineral-admixtured concrete. Cement and Concrete Research, 34(12) pp. 2329-2336.

Ramazan D. & Rüstem G. (2006) Production of high strength concrete by use of industrial by-products. Building and Environment, 41(8) pp. 1124-1127.

Ravindrarajah R.S., Steward M. & Greco D. (2000) Variability of recycled concrete aggregate and its effect on concrete properties: a case study in Australia. International workshop on recycled concrete, Australia.

Ravindrarajah R.S. & Tam T.C. (1985) Properties of concrete made with crushed concrete as coarse aggregate. Magazine of Concrete Research, 37, p. 130.

RILEM (1992) Recycling of demolished concrete and masonry. London, E & FN Spon.

RILEM (1994) RILEM TC 121 DRG Recommendation: specification for concrete with recycled aggregates. Materials and Structures, 27(173) pp. 557-559.

Rockliff D. (1996) Low-grade quarry products, reclaimed aggregates and inert wastes - their use in unbound mixtures for road pavements. Waste Management, 16(1-3) pp. 83-85.

Rock Products (2009) Cement production. [Internet] Available from: <http://rockproducts.com/news/rock_cement_production_reach/ > [Accessed on 10 January 2009].

Saak H.M.J. & Shaf S.P. (1999) Characterization of the rheological properties of cement paste for use in self-compacting concrete. In: Proceedings of 1st International RILEM Symposium on SCC. Stockholm, Sweden: RILEM publications, SARL, pp. 83-84.

Sagoe C.K.K. & Brown T. (1998) Guide for specification of recycled concrete aggregate for concrete production. Commonwealth Scientific Information and Research Organization, London, UK.

Sagoe C.K.K., Brown T. & Taylor A.H. (2001) Performance of concrete made with commercially produced coarse recycled concrete aggregate. Cement and Concrete Research, 31(5) pp. 707-712.

Sakai E., Hoshimo S., Ohba Y. & Daimon M. (1997) The fluidity of cement paste with various types of inorganic powders. In: Proceedings of the 10th International Conference on the Chemistry of Cement, Stockholm, Sweden, p.8.

Şakir E. (2005) Effect of retempering with superplasticizer admixtures on slump loss and compressive strength of concrete subjected to prolonged mixing. Cement and Concrete Research, 35(5) pp. 907-912.

Salem R.M. & Burdette E.G. (1998) Role of chemical and mineral admixture on physical properties and frost-resistance of recycled aggregate concrete. ACI Materials Journal, 95(5) pp. 558-563.

Salomon M.L. & Paulo H. (2004) Durability of recycled aggregates concrete: a safe way to sustainable development. Cement and Concrete Research, 34(11) pp. 1975-1980.

Sear K.A.L. (2005) Should you be using more PFA? In: Proceedings of Dundee University Conference - Global Construction - Ultimate Concrete Opportunities. 5-7 July, Concrete Technology Unit, University of Dundee.
[Internet] Available from:
<http://www.ukqaa.org.uk/Papers/DundeeJuly2005ShouldYouBeUsingMorePFA.pdf>
[Accessed on 11 June 2006].

Sebök T. & Stráněl O. (2004) Wear resistance of polymer-impregnated mortar and concrete. Cement and Concrete Research, 34(10) pp. 1853-1858.

Shafiq N., Nuruddin F.M. & Kamaruddin I. (2006) Comparison of engineering and durability properties of fly ash blended cement concrete made in UK and Malaysia. In: Proceedings of Cement and Concrete Science Conference No 26, Sheffield, UK, 11 September, 106(6) pp. 314-318.

Sherwood P.T. (1995) The use of waste and recycled materials in roads. Proceedings of the Institution of Civil Engineers, Transport, 111(2) pp. 116-124.

Shokry R. & Siman A. (1997) Research on an alternative method for reclaiming leftover concrete. Concrete International, 19(7) pp. 53.

Smith D. (1988) Call for greener building regulations. New Civil Engineer, 5(1) pp. 22-25.

Sofiane A. (2006) Setting time determination of cementitious materials based on measurements of the hydraulic pressure variations. Cement and Concrete Research, 36(2) pp. 295-304.

Speare R.S. (1995) Recycling of structural materials. The Structural Engineer, 73(13) pp. 220-222.

Su N., Hsu K.C. & Chai H.W. (2001) A simple mix design method for self-compacting mortars. Cement and Concrete Research, 31(12) pp. 1799-1807.

Suneel N.V. (2004) The advances and barriers in application of new concrete technology. In: Proceedings of the International Workshop on Sustainable Development and Concrete Technology. 20–21 May, Beijing, China. Iowa State University. [Internet] Available from: > [Accessed on 15 July 2006].

Swamy R.N., Sami A.R.A. & Theodorakopoulos D.D. (1983) Early strength fly ash concrete for structural applications. ACI Materials Journal, 9, pp. 414-423.
Tahir C. & Khaled M. (1996) Effect of crushed stone dust on some properties of concrete. Concrete and Cement Research, 26(7) pp. 1121-1130.

Tarun R.N., Shiw S., Singh S.S. & Hossain M.M. (1996) Enhancement in mechanical properties of concrete. Cement and Concrete Research, 26(1) pp. 49-54.

Tavakoli M. & Soroushian P. (1996) Strength of recycled aggregate concrete using field-demolished concrete as aggregate. ACI Materials Journal, 93(2) pp. 182-190.

Tayyeb A., Shazim A.M. & Humayun O. (2009) Production of low cost self compacting concrete using bagasse ash. Construction and Building Materials, 23(2) pp. 703-712.

Topcu B.I. & Guncan F.N. (1995) Using waste concrete as aggregate. Cement and Concrete Research, 25(7) pp. 1385-1390.

Topcu B.I. & Ugurlu A. (2003) Effect of use of mineral filler on the properties of concrete. Cement and Concrete Research, 33(7) pp. 1071-1075.

Tsung Y.T., Yuen Y. & Chen L.H. (2006) Properties of HPC with recycled aggregates. Cement and Concrete Research, 36(5) pp. 943-950.

Turcotte D.L. (1993) Fractals and chaos in geology and geophysics. Cambridge, Cambridge University Press.

Van A. (1996) Recycling of concrete at a precast concrete plant. Concrete Precasting Plant Technology, 6, pp. 91-101.

Vanchai S., Jaturapitakkul C. & Kraiwood K. (2007) Influence of pozzolan from various by-product materials on mechanical properties of high-strength concrete. Construction and Building Materials, 21(7) pp. 1589-1598.

Vieira C.M.F., Soares T.M., Sánchez R. & Monteiro S.N. (2004) Incorporation of granite waste in red ceramics. Materials Science and Engineering, 373(12) pp. 115-121.
Vivian W.Y.T. & Tam C.M. (2007) Crushed aggregate production from centralized combined and individual waste sources in Hong Kong. Construction and Building Materials, 21(4) pp. 879-886.

Wainwright P.J. (1993) Modifying the performance of concrete made with coarse and fine recycled aggregates. In: Proceedings of the Third International RILEM Symposium on Demolition and Reuse of Concrete and Masonry, 24–27 October Odense, Denmark, E&FN Spon, pp. 319-330.

Walsh I.D. (1997) Meeting the clients' needs. In: Proceedings of the CSS Conference on Sustainable Road Maintenance. Specifications for Recycled Aggregates in Roads. 18 November, Leamington Spa, UK, Building, 3 pp. 45-49.

Waste (2008) Waste Disposal. [Internet] Available from: <http://www.ace.mmu.ac.uk/eae/Sustainability/Older/Waste_Disposal.html> [Accessed on 25 November 2008].

Watson K. (1993) Rocky road to recycling. Surveyor, 180(52) pp. 17-18.

Webb C. (1999) Roll out the waste heaps. Surveyor, 176, pp. 51-60.

White E.R. (2005) Principles and practice of soil science: the soil as a natural resource, 4th edition, Wiley-Blackwell, London.

Wilbert L. & Gordon L. (1998) High volume fly ash concrete. In: Proceedings of the 6th CANMET/ACI/JCI International Conference on Fly ash, Silica Fume, Slag and Natural Pozzolan in Concrete, Bangkok, Thailand, pp. 1021-1040.

Wilson A., Francisco A.V. & Ana M.S. (2006) Using ornamental stone cutting rejects as raw materials for red clay ceramic products: Properties and microstructure development. Materials Science and Engineering, 435, pp. 606-610.

Wrap (2006) Use of the demolition protocol for the Wembley development. Wrap-Report No AGG0078. [Internet] Available from: <http://www.wrap.org.uk/downloads/WRAP_Case_Study_SAC_formatted_050307.a4024871.4649.pdf> [Accessed on 12 February 2006].

Wrap (2007) Performance related approach to use of recycled aggregate. [Internet] Available from: <http://www.aggregain.org.uk/news/performance_related.html> [Accessed on 25 August 2008].

Xiao J.-Z., Li J.-B. & Zhang C. (2006) On relationship between the mechanical properties of recycled aggregate concrete: an overview. Materials and Structures, 39(6), pp. 655-664.

Xie Y., Liu B., Yin J. & Zhou S. (2002) Optimum mix parameters of high-strength self-compacting concrete with ultrapulverized fly ash. Cement and Concrete Research, 32(3) pp. 477-480.

Yijin L., Zhou S., Yin J. & Gao Y. (2008) The effect of fly ash on the fluidity of cement paste, mortar and concrete. [Internet] Available from: < http://www.cptechcenter.org/publications/sustainable/lifluidity.pdf> [Accessed on 14 March 2008].

Zhou F.P., Lydon F.D. & Barr B.I.G. (1995) Effect of coarse aggregate on elastic modulus and compressive strength of high strength concrete. Cement and Concrete Research, 25(1) pp. 177-186.

Zhu W. & Gibbs J.C. (2005) Use of different limestone and chalk powders in self-compacting concrete with ultrapulverized fly ash. Cement and Concrete Research, 35(8) pp. 1457-1462.

Zilch K. & Roos F. (2001) An equation to estimate the modulus of elasticity of concrete with recycled aggregates, Civil Engineering ,76(4) pp. 187–191.