The University of Nottingham

The Properties of Recycled Precast Concrete Hollow Core Slabs for Use as Replacement Aggregate in Concrete

by

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Abstract

The dumping cost of wasted concrete including the rejected units in precast concrete plants is expected to keep rising as the production increases. The waste material from precast concrete hollow core floors (hcu) is high grade and uncontaminated material. This research work was carried out to investigate mainly the strength and other engineering properties of high strength concrete made with recycled concrete aggregate derived from rejected hcu. Three major categories (based on a questionnaire) were investigated: (i) Type of crushers and the crushing method, (ii) The properties of RCA output from these crushers, (iii) The performance of fresh and hardened concrete, including prestressed concrete, with these RCA.

The input material for the crushers was from the same origin of disposed hcu's. The waste concrete was crushed to –14 mm using three different types of crushers – the cone, impact and jaw crushers. The recycled material was separated into fractions of 14 mm, 10 mm and – 5 mm, and tested for physical and mechanical properties relevant to use in concrete. Concrete was then made using zero (control mix), 20% and 50% replacement of recycled coarse (RCCA), recycled fine (RCFA) and mixed (RCCA+RCFA) aggregates.

All three crushers produced acceptable shape and strength of RCCA. Some properties are competitive to that of natural limestone aggregate. RCFA was much coarser than river gravel and just complied with the British Standard coarse grading limits. The impact crusher performed best with regard to most aggregate properties, e.g. flakiness, strength and water absorption, but has a disadvantage in producing a large amount of fine-to-coarse RCA. Concerning shape and strength, RCA showed similar properties, and in some cases better, than the conventional limestone aggregate.

The water absorption for RCA is 3 to 4 times greater than the natural aggregates. For that reason an extra amount of water (called free water) will be added to the mix to compensate the water absorptions for aggregates. Some proportions of this extra added water may not be absorbed by the aggregates and will float to interrupt the design W/C ratio and caused it to increase.

The slump value of fresh concrete made with RCA varied widely depending on the percentage and type of replacement, and the type of crusher. The compaction factor of fresh concrete made with RCA was more consistent and logical.

Compressive strength of concrete made with RCA were generally within ±5 N/mm^2 of the control. For tensile strength, RCA showed similar performance to that of natural limestone. The SS density of concrete with RCA is lower than that of the control concrete and is lower if the replacement percentages increase. Using RCFA causes higher bleeding rate and considerably reduces density and strength, and the severity increases as the replacements of RCFA increases. Using natural limestone aggregates with RCFA will minimize this poor behaviour and maintain the strength to certain extent. However joining RCCA with RCFA will not limit the poor behaviour and is not recommended.

For bonding reinforcing bars most methods indicated that high replacement (100%) of RCA cause some reduction in bond strength. In pretensioning wires the RCA concrete had a better performance in bond but some reduction was still reported. Prestressed X-shape beams were used to assess the effects of using of RCA on the performance of hollow core slabs. For 20% RCCA replacements, the prestressing loss, deflection and X-beam flexure crack failure were similar to the standard X-beam, at least and within the design limit. However at higher replacements (50%) some deterioration starts to reveal and the effects are even greater when using a combination of RCCA and RCFA.

Declaration

I declare that this thesis is the result of my own work. No part of this thesis has been submitted to another university or any other educational establishment for a degree, diploma or other qualification (except for publication)

Basem E Marmash

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In memory of my brother and my father Yahia Marmash and Ezzat Marmash, may peace and blessings of God be upon them

Notation

A_c	Concreat area
A_p	Cross-sectional area of prestressing strand
E_c	Youngs modulus of concrete
E_s	Youngs modulus of steel
V_c	Concrete contribution to shear capacity
V_{cr}	Cracking shear capacity
d	effective depth
f_{ct}	concrete tensile strength
$f_{ts} : \sigma_t$	tensile splitting strength
f_{cu}	cube compressive strength
f_{ctk}	characteristic concrete tensile strength
f_{pu}	ultimate tensile strength of prestressing strand
f_t	compressive strength at top of section due to prestress
f_{bc}	final stress in the wires after loss
σ r	radial Stress
σ po	stress at the wire
ε_c	concrete strain
M_{cr}	moment where the flexure cracks values occurred
h	height of section
l_t	transfer length of prestressing stand/wire
x	depth to neutral axis
z	moment lever arm
l_d	development length of prestressing strand/wire
I	Second-moment of area
Z_b	Section modulus(bottom)
Z_t	Section modulus(top)
η	bar friction coefficient value
Δc	average distance between cracks measured.
δ	deflection

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1 Research Outline

1.1 Introduction

There is a considerable concern over the use of recycled concrete as a new source of aggregates. The source of raw materials for building industries in many countries, for instance some Western European countries, is clearly changing as the environmental regulations are becoming more restrictive. The dumping cost of either the residues of fresh and hardened concrete is expected to keep rising. Some studies, Waste Regulation Authority - London, carried out before the year 1996 revealed that over 2 billion tonnes of waste are generated in European Union each year from a population of 342 million [1]. It is estimated that 190 million tonnes of construction and demolition waste arises each year from only 12 countries of the European Union, of which 95% is landfill and just 5% is recovered by the year 1996 [1]. By the year 2007 Eurostat, the Statistical Office of the European Communities revealed the member states with the highest share of municipal waste landfilled were Bulgaria (100%), Romania (99%), Lithuania (96%), Malta (93%) and Poland (90%), however the UK achieves the ninth best recycling rate in Europe with 57% of its municipal solid waste sent to landfill. In the year 2009, Paine [2] explained that the concrete industry consumes approximately 40% of the total worldwide construction aggregate production. However, at present its use of recycled aggregates is marginal, with possibly as few as 3% of all aggregates used being from recycled sources. One of the main reasons for this is a misconception that they are inferior aggregates. The total waste arising in the UK are around 272 million tonnes per annum of which the construction sector accounts for 32% of this waste, make it the largest single source of waste [3]. On the other hand, the fact remains that about 83% of aggregates used in the UK are from primary land-won sources [1]. Although the economic cycles have an impact on the aggregates usage as it happened in the nineties where the total natural aggregate usage in UK has reduced from 300 million tonnes in 1989 to an estimated 218 million tonnes in 1997 due to the recession, but it is estimated that the annual consumption of aggregates in the UK would increase from the 220 million tonnes used in 1991 to about 400 million tonnes by 2011 [4]. It is obvious the demand will increase but also there is good potential to increase resource efficiency in construction and reduce waste. Although the recycling of construction waste has increased but rates of land filling from site construction waste still appear to be high and there is scope for improved performance [3]. To stimulate diversion from landfill, the UK government proposed a target of halving the amount of construction waste going to landfill by 2012 as a result of waste reduction, re-use and recycling programme [3].

From those points, the need for recycling concrete as a new source of aggregate was originated and, therefore, some works have been carried out to investigate the strength and other engineering properties of concrete made with RCA. Such works had been carried out in various countries particularly in Belgium, Netherlands and Japan, mainly in the recycling of waste building material in concrete, which often derives from varied and unknown constituents.

Almost all reported that the insufficient performance of concrete with RCA is mainly related to the attached old cement paste (mortar) and to some deficient properties of RCA, e.g. shape, texture, porosity and absorption. The angularity, harshness, surface texture and other similar properties of RCA are likely to be dependent on the method that is used to crush the waste material into aggregate size particles, typically 20 mm down to 5 mm for coarse aggregate (The crushing plant and their different methods will be explained later). Furthermore, porosity and absorption of RCA are dependent on aggregate origin as well as on the amount of mortar attached to the RCA particles; these in turn could depend on the method of crushing.

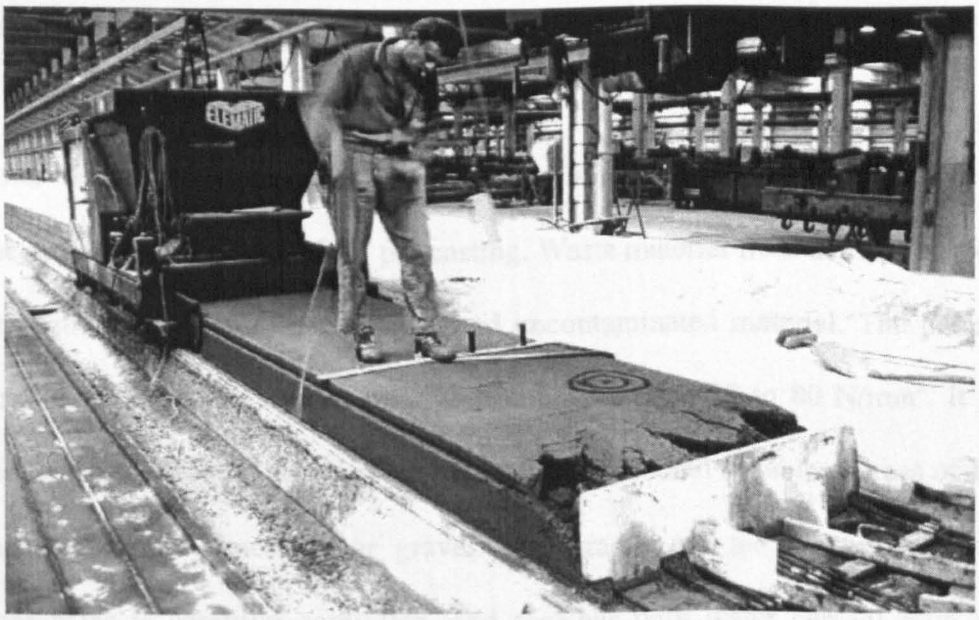


Figure 1-1: End line waste of hollow core slab units

In addition, the quality of recycled fine aggregate (RCFA) could also be related to the method of crushing. Therefore it is necessary to investigate how far the crushers themselves and the method affect the properties of RCA and the performance quality of concrete containing RCA. Although much of the previous work has been carried out on demolished structures with different constituents, less work has been done on waste concrete generated from precast concrete industries. For example, the precast concrete hollow core floor industry produces a considerable amount of waste elements, due mainly to the manufacturing processes and in part to natural wastage at the ends of the casting beds. Figure 1-1 shows how prestressed hollow core units (hcu) are manufactured by extrusion or slip-forming through a machine on long beds, typically 100 m in length x 1.2 m wide. Waste material is made at the beginning and end of each bed, typically 0.3 to 0.4 m^3 per casting.

After de-tensioning, the units are cut to length where waste material is therefore made of up to 0.5 m^3 per casting. Waste material from hcu has known properties and is of a high quality and uncontaminated material. The parent concrete is hard and of compressive strength between 50 to 80 N/mm^2. It is manufactured from Portland cement, and from clean and reliable sources of 10 mm to 14 mm limestone or gravel. The grading of the coarse and fine aggregates is carefully controlled, and together with water cement ratio of around 0.3 the resulting concrete is of a high density and low porosity.

1.2 Objectives

The main objectives of this research is to carry out a systematic investigation into the influence of recycled aggregates, derived from hcu, using different crushing methods on the performance of concrete and try to established whether any relationship between crushing method, properties of crushed recycled aggregate (RCA Crusher's output) and the performance of concrete (including precast) with RCA. Therefore the following criteria will be included:

1. Crushing methods, taking into consideration the type of crusher, its specification and performance.
2. Properties and characteristics of the RCA that has originated from various grades of disposed prestressed concrete units.
3. Performance and properties of fresh and hardened concrete (including prestressed concrete) made with these RCA.

1.3 Outline of Thesis

The three major objectives, mentioned earlier, were investigated and approached as outlined in the following chapters:

Chapter 2:

This chapter points out most previous research work related to recycling in the concrete industry. It summarises their findings and recommendations for specifying and using RCA.

Chapter 3:

This chapter attempts to clarify and to prioritise the obstacles that face the concrete industry in using RCA. It points to the questionnaire that was sent for this purpose and its outcome is discussed.

The three crushing methods, namely the *Jaw*, *Cone* and *Impact*, and their effect on recycled aggregate are discussed. An explanation of these crushing machines is given. The input materials were from the same origin of commercial hcus manufactured by Richard Lees Ltd (now Tarmac) using the *Spiroll* extrusion technique.

Chapter 4:

This chapter focuses on the properties of RCA, separated into coarse (10-14 mm) and fine (≤5 mm) fractions, which are important to the reintroduction of RCA in hcu production, namely grading, water absorption, density, shape and strength.

The properties relating to each of the crushers were investigated and compared with each other as well as to the BS specifications for conventional natural crushed aggregate.

Chapter 5:

In this chapter, the RCA were tested in concrete mixes in which the aggregates in a 'reference' mix were substituted with varying proportions of RCA. The chosen percentage replacement (by mass) was 20% and 50% - the former represents a typical limit for RCCA proposed in P.I.T. project [5], and the latter is made deliberately large in order to investigate the sensitivity of the important mechanical and physical properties. Mix design and concrete consistency, i.e. workability, were investigated and compared with each other related to the crushers as well as to the standard concrete with conventional crushed limestone aggregate. In this research the term workability is used throughout the thesis instead of consistency.

Chapter 6:

This chapter covers the mechanical properties of concrete with the recycled aggregate (a year old of hardened hcu) obtained using three different crushers and compares the result with that of the control mix. Density and concrete strength (compressive, flexural and tensile splitting strength) were investigated and compared with each other as well as with the standard concrete.

Chapter 7:

This chapter presents the investigation into the effect of using RCA on the bond between concrete and both the reinforcing bars and prestressing wires. The development of bond stress was deduced by measuring the distance between tension cracks in prisms where its reinforcement bar subjected to axial tension. The tension versus elongation was considered and discussed.

Chapter 8:

This chapter focuses on how the RCA affects prestressed concrete through a number of 3-point bending tests performed on x-shaped sections. The x-shape was chosen because it closely simulates the rounded webs of an extruded hollow core slab. The aim was to assess the effects of adding proportions of RCA on the flexural performance of prestressed concrete. The strain losses and deflections were recorded and discussed. The strain on the concrete surface and in the wires were monitored and compared to standard concrete.

Chapter 9:

The conclusions comment on the basic properties of the aggregates resulting from the crushing method, and how they translate into parallel properties in concrete. The work is shown to be largely experimental; a major focus of the study is the relative, rather than absolute, differences in the behaviour of RCA and natural aggregate concrete.

2 General Background

2.1 Recycled Concrete Aggregate

In general recycled concrete aggregates falls into two categories, (i) recycled aggregates derived from concrete waste only; it is known as recycled concrete aggregate (RCA) while the other type (ii) recycled aggregate obtained from a wide-ranging of mix materials i.e. buildings rubbles comprising concrete, bricks and other building waste, this is referred to as recycled aggregate (RA); these definitions are in compliance with BS 8500-2 which specifies constituent materials and concrete. However, because of the low proportion of masonry permitted, EN 12620, the European standard for aggregates has included a new classification for recycled aggregates in an attempt to promote the use of material containing less crushed concrete but this is out of this research work aims. Many researches have investigated the feasibility of using the recycled aggregate in concrete. They have considered both the physical and mechanical properties of RCA and RA together in a manner that might affect the concrete properties and its performance. It should be noted that RCA obtained from prestressed concrete waste (e.g. hcu), is of high grade uncontaminated material and tends to be brittle with sharp edges, which is differ from RCA derived from ordinary concrete waste. The following are the most relevant research results; more emphasis is put on aggregate of concrete waste (RCA) including that of high strength concrete, which can be related to this research work.

2.1.1 Physical properties of Recycled Aggregate

2.1.1.1 Grading

Proper grading of aggregate is necessary to produce well compact and dense concrete. Hansen [6] compared the grading of an average jaw crusher product with ASTM grading requirements. His data were obtained from Danish and Japanese investigations as follows. The grading of recycled fine aggregates (RCFA) produced by means of a jaw crusher, Figure 2-1, are somewhat coarser than the lower limit of ASTM grading requirements [7] and some are even lower than the lowest permissible grading limit of zone 1 sand in BS 812-103 [8]. No further details about the jaw crusher's setting mentioned in the report. For the RCCA, Figure 2-2, showed that they are within the ASTM grading standards. He concluded that both coarse and fine aggregates could be brought to ASTM grading standard by slightly adjusting the opening of the jaw crusher. Mulheron [9] indicated the influence of the crusher type on the grading of RCA. The fundamental difference between jaw and impact crusher lies in the method by which the material is crushed. He referred to Boesman [10] who noted the effect the crushing machine has on particle shape and size distribution, i.e. impact crushers produce more angular particles than jaw crushers, and they produce twice the amount of fines than jaw crushers for the same maximum size of RCCA. It appears that Hansen and Mulheron find opposing results, with that latter more satisfied with the jaw crusher. Boesman notes that the type of crusher does not significantly affect the physical properties of recycled aggregates, such as water absorption density and abrasion loss percentage. This fact will be noted later in this chapter.

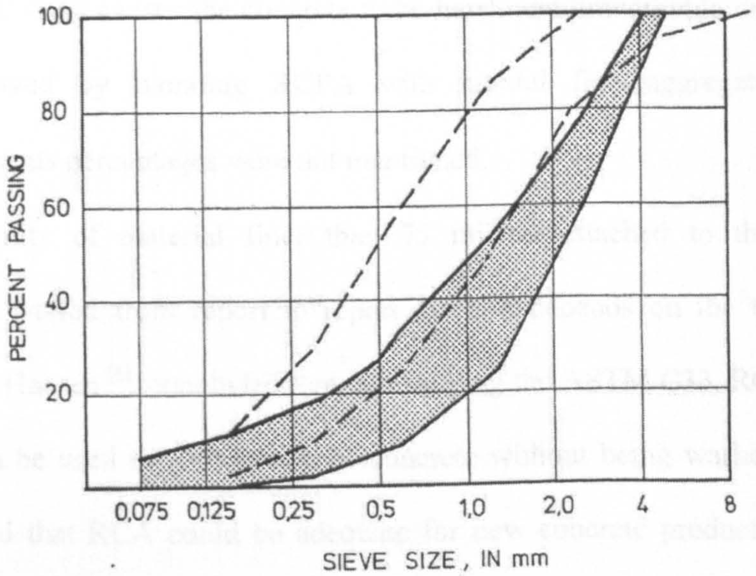


Figure 2-1 Range of grading of fine recycled aggregate (< 4 mm) obtained when 25-30 mm maximum size coarse recycled concrete aggregates are produced by jaw crusher in one pass (from Hansen [6]) Dashed lines show ASTM grading limits.

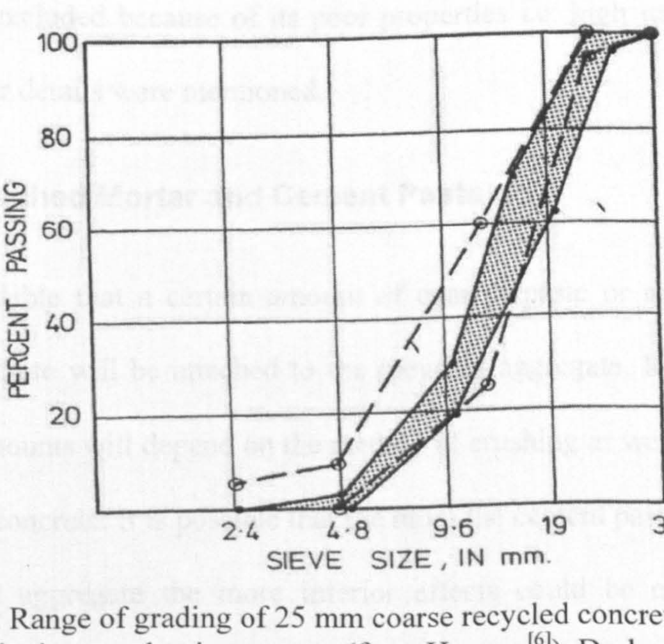


Figure 2-2: Range of grading of 25 mm coarse recycled concrete aggregates produced by jaw crusher in one pass (from Hansen [6]). Dashed lines show ASTM grading limits.

2.1.1.2 Particle Shape and Surface Texture

Hansen [6] has found that RCFA is coarser and more angular than natural fine aggregate. This causes the concrete to be harsh and unworkable, which could be improved by blending RCFA with natural fine aggregates (NFA). Replacements percentages were not mentioned.

The quantity of material finer than 75 micron attached to the recycled aggregate varied from report to report since it depends on the type of the concrete. Hansen [6] concluded that, considering the ASTM C33, RCA in most cases can be used for production of concrete without being washed. He also mentioned that RCA could be adequate for new concrete production only if particle size below 2 mm were screened out.

Hansen's conclusions are based on Morlion [11], who reported that RCCA and natural sand were used for concrete production at the large recycling plant but RCFA was excluded because of its poor properties i.e. high water absorption but no further details were mentioned.

2.1.1.3 Attached Mortar and Cement Paste

It is unavoidable that a certain amount of cement paste or mortar from the original concrete will be attached to the recycled aggregate. It is more likely that these amounts will depend on the method of crushing as well as the type of the original concrete. It is possible that the more the cement paste is attached to the recycled aggregate the more inferior effects could be encountered on certain properties of concrete with these RCA.

Hansen and Naured [12], on the basis of the results of an investigation by Hedegaard [13], measured the quantity of attached mortar by casting recycled aggregate and red-coloured cement paste into cubes. These cubes were sliced and polished to determine the distinction between the red cement matrix and the old matrix attached to the recycled aggregate. Then the volume percentage of old mortar was determined by means of a linear traverse method (similar in principle to the method described in ASTM C 457-98 [14] - Practice for microscopical determination of air void content and parameters of the air void system in hardened concrete. They reported that the volume percentage of mortar attached to natural gravel particles to be between 25% and 35% for 16-32 mm RCCA, around 40% for 8-16 mm RCCA and around 60% for 4-8 mm RCFA. More details are given in Table 2-1. The larger amount of mortar in RCFA is because the volume of a small particle is small in comparison to the larger one, yet the quantity of mortar adhered to the particles is about the same. This suggests problems with WA for small particles will be encountered. They also explained that the volume percentage of old mortar attached to RCA does not vary much even for widely different water to cement ratios of the original concrete.

Hansen [12] refers to B.C.S.J. [15] who measured the amount of old hydrated cement paste attached to recycled aggregate by immersing the particles in a dilute solution of hydrochloric acid at 20°C. The weight loss due to dissolution of cement during the test is the amount of cement paste attached to the RCA. He concluded that the amount of cement paste increases as the particle size of RCA decreases; see Figure 2-3. Approximately 20% of cement paste is attached to 20-30 mm of aggregates while the 0-0.3 mm filler fraction of recycled fine aggregate contains 45-65% of old cement paste.

Table 2-1: Properties of natural gravel and RCA (Hansen [1])

Type of Aggregate	Size fraction (mm)	Specific Density SSD (kg/m^3)	Water absorption%	L.Angeles abrasion Loss%	A.I.V BS %	Volume% of mortar attached to natural Gravel Particles
Original Natural Gravel	4-8 8-16 16-32	2500 2620 2610	3.7 1.8 0.8	25.9 22.7 18.8	21.8 18.5 14.5	0 0 0
(H) Recycled Aggregate (W/C=0.4)	4-8 8-16 16-32	2340 2450 2490	8.5 5.0 3.8	30.1 26.7 22.4	25.6 23.6 20.4	58 38 35
(M) Recycled Aggregate (W/C=0.7)	4-8 8-16 16-32	2350 2440 2480	8.7 5.4 4.0	32.6 29.2 25.4	27.3 25.6 23.2	64 39 28
(M) Recycled Aggregate (W/C=1.2)	4-8 8-16 16-32	2340 2420 2490	8.7 5.7 3.7	41.4 37.0 31.5	28.2 29.6 27.4	61 39 25
(M) Recycled Aggregate (W/C=0.7)	< 5	2280	9.8	-		

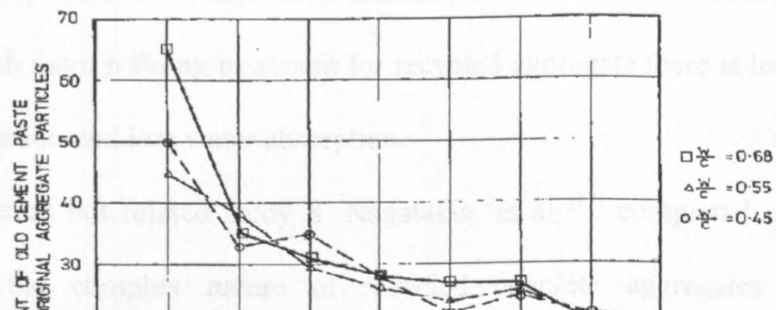


Figure 2-3: Weight percentage of cement paste adhering to original aggregate particles in recycled aggregate produced from original concrete with different water cement ratios (Hansen [6])

Kopayahi and Kawano [16] investigated the influence of refining treatment (removal of cement paste) on RCA. They related the differences in the properties between the original and the recycled aggregate to the adhesion of cement paste. They found that the water absorption of RCCA derived from high strength concrete without refining treatment to be 4.0% on average while for the same RCCA but with high refining treatment to be 1.57%, where for the original aggregate it was 0.70%.

Similar findings are reported for RCFA where he reported that the water absorption for RCFA derived from high strength concrete with a minimum refining treatment to be 7.80% while that with a high refining treatment to be 6.18%, both compared to original RCFA of 1.79%. The amount of adhering cement paste reported to reached up to 20% before any refining treatments. They concluded that a linear relationship was outlined between the amount of water absorption and the amount of cement paste adhered to the RCA. In other words with more refining treatment for recycled aggregate there is less cement paste adhesion and less water absorption.

In a different but related study S. Nagatakia, et al [17] completed a study to evaluate the complex nature of recycled concrete aggregates that are susceptible to damage due to recycling. Using fluorescent microscopy and image analysis in their studies they reported that the adhered mortar is not always the primary parameter determining the quality of the recycled coarse aggregate; they found Sandstone coarse aggregate originally had defects in the form of voids and cracks. Their studies were based on laboratory-produced concretes as the source of the recycled aggregates. Three different water–

cement ratios were used; low (0.35), medium (0.45) and high (0.63) with 28 days compressive strength of 60.7, 49 and 28.3 N/mm^2. Combination of jaw crusher and impact crusher were used to crush the concrete and then the crushed recycled aggregate were processed with a mechanical grinding equipment to produce recycled aggregates with highest attached mortar and other with lowest attached mortar. They concluded that both compressive strength and tensile splitting strength of the concrete made with the recycled coarse aggregates originated from high (60N/mm^2) and medium (49N/mm^2) quality source concrete gave noticeably higher strength values than that of the original aggregate concrete. They explained that the elimination of the friable and porous aggregate particles during recycling and crushing process created almost micro defect-free recycled coarse aggregates with a high level of integrity resulting in better mechanical performance.

Juan et al [18] carried out a study to obtain experimental relationships between the attached mortar content and recycled aggregate properties including aggregates fraction size, absorption, Los Angeles abrasion, Sulphate content. To establish the mount of mortar attached to the recycled aggregates he used the thermal method because to his view it can be used for all kind of aggregates (including limestone) and it is easier than other methods. He explained the method is started by preparing the recycled aggregate sample (mi) and then immersed in water for 2 hours to ensure the attached mortar are completely saturated. Next, the sample is dried in a temperature of 500 C° for 2 hours. Then, the sample is immersed into cold water. This sudden cooling causes stress and cracks in the mortar and then can be easily removed. After defining the amount of attached mortar He carried out tests to establish recycled aggregate properties and found that the amount of mortar attached to fine fraction is around 33–55% of its weight which is higher than to coarse fraction, 23–44%. He also found that higher contents of attached mortar cause higher values of water absorptions, Los Angeles abrasion and Sulphate content. He concluded that recycled aggregates with mortar content under 44% could be used of structural concrete. With such amount of attached mortar the recycled aggregates are expected to have bulk specific density higher than 2160 kg/m^3, water absorption lower than 8% and Los Angeles abrasion loss under 40%. As such with this quality of recycled aggregate can be produced controlling original concrete strength, over 25 N/mm^2.

2.1.1.4 Density

Hansen [12] concluded that the surface saturated density (SSD) of RCA is lower than the SSD of the original aggregate ranging from 2340 kg/m^3 (for 4-8 mm material) to 2490 kg/m^3 (for 16-32 mm). This is due to a relatively lower density of the old mortar that is attached to original aggregate particles. For the same cement and original aggregate the density of RCA does not vary much even for widely different water to cement ratios of original concrete. His conclusion was based on several reports, including Hasaba *et al* [19], B.C.S.J. [15] and others. Hasaba [19] reported that the surface saturated density (SSD) of 25-5 mm RCCA was about 2430 kg/m^3 independent of the quality of original concrete.

The SSD of recycled fine aggregate below 5 mm was 2310 kg/m^3, while the density of the original coarse and fine aggregate were 2700 kg/m^3 and 2590 kg/m^3, respectively.

B.C.S.J [15] reported that from a wide range of original concrete the dry density of RCCA varied between 2120 kg/m^3 and 2430 kg/m^3 and the SSD between 2290 kg/m^3 and 2510 kg/m^3. For RCFA the dry density varied between 1970 kg/m^3 and 2140 kg/m^3 while the SSD ranged between 2190 kg/m^3 and 2320 kg/m^3. It may be concluded that the density of recycled aggregates is lower than the density of the original aggregate due to a relatively low density of old mortar which is attached to original concrete particles. This is particularly clear for RCFA which shows higher amount of old mortar attached causes higher water absorption and lower density, which leads to a poorer performance of concrete as will be seen in later chapters.

However Mulheron and O'Mahony [20], narrowed down their work to two types of RCA and they showed the advantages of RCA from crushed concrete to that from debris. They investigated the physical and mechanical properties of two different types of recycled aggregates, one obtained from crushed concrete and the other from well-graded clean debris, and found that the specific gravity of the RCA was about 6% lower than that of natural gravel, while for recycled debris aggregate were about 18% lower.

Kopayahi and Kawano [16] reported that the specific gravity of both RCCA and RCFA were lower than for natural aggregate. They also found the important conclusion that the higher the refining treatment (a process of removing cement paste from RCA) the quality of RCA improves, and this leads to little differences in the density between RCA and natural aggregate.

2.1.1.5 Water Absorption

It was concluded by most reports that water absorption for RCA is higher than that for natural aggregate. Hansen and Narud [12] reported that, regardless of the quality of original concrete, water absorption for RCCA ranged from 8.7% for 4-8 mm to 3.7% for 16-32 mm. Those findings were confirmed by Mulheron and O'Mahony [20] who reported that the water absorption for RCCA ranged from 5.3% to 8.3%. Hasaba [19] also found that water absorption for 25-5 mm RCCA to be 7%, and for RCFA below 5 mm to be 11% regardless of the quality of the original concrete. Kreijger [21] shows an inverse and non-linear relationship between water absorption and density of recycled aggregates, as shown in Figure 2-4.

Hansen [6] concluded that the higher water absorption for RCA is related to the higher porosity and water absorption for the old mortar attached to it. He indicated that it is more difficult to determine the water absorption for fine recycled aggregate than coarse recycled aggregate. Hansen and Marga [22] reported some difficulties and disadvantages in studying RCFA. They explained that the ASTM C128 [23] is inadequate and highly inaccurate to be used to measure the specific gravity and water absorption of RCA. They explained that it is difficult to assess when RCFA are in saturated and surface dry condition. They explained that the RCFA becomes cohesive and attached to surfaces, i.e. does not run freely.

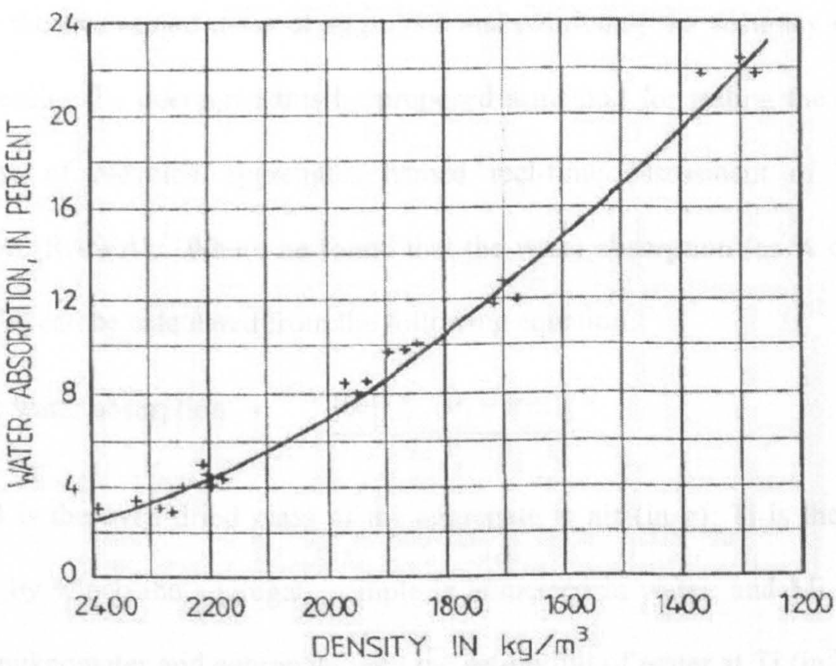


Figure 2-4: Water absorption as a function of density of recycled concrete aggregate (Hansen[6] from Kreijger[21])

Tam, et al [24] suggested that the traditional testing approach for water absorption cannot give accurate results for RA, based upon which, errors in concrete mix designs may result. He explained that the inaccuracy is related to the cement paste attached to recycled aggregates might detach from the mass during sample preparation. He added that drying at 105 ± 5 C° to obtain the oven-dried mass of aggregate may remove water chemically incorporated in the crystal structure of compounds in the mortar attached to the aggregate. Removing the crystallized water will give a misleading level of water absorption of aggregate. He added that soaking time before reaching full saturation for recycled aggregate varies from the conditions of surface cement pastes on aggregate. He also explained the BSI approach requires surface-drying the aggregate with a cloth or towel which may cause detachment of some cement paste sticking on the surface of aggregate thus significantly reducing the oven-dried mass of aggregate and restricting the accuracy of the testing result. To overcome this he proposed a method for testing the water absorption of recycled aggregates named real-time assessment of water absorption (RAWA). Where he found that the water absorption (as % of dry mass) at Ti can be calculated from the following equation:

$$\text{The water absorption} = \frac{100\left[\sum_{j=1}^{i}(M_j - M_{j-1})\right]}{B}$$

Where B is the oven-dried mass of the aggregate in air (in g); Ti is the time intervals by which the aggregate sample is immersed in water; and Mi is the mass of pyknometer and aggregate with the set-up full of water at Ti (in g). he also reported that the total water absorption (as % of dry mass) of the saturated surface-dried aggregate at Ts can be calculated from the following equation:

$$\text{The total water absorption} = \frac{100[(M_s - M_0)]}{B}$$

Where Ms is the mass of the whole set-up full of water at Ts; and M_0 is the mass of the set-up at T_0.

Merlet and Pimient [25] reported that partial substitution of recycled fine aggregate by natural sand will improve the concrete properties and they added pre-moistening the recycled aggregate will improve the mechanical properties and decrease the drying shrinkage, no details in how the RCA were pre-moisten. Hansen [6] concluded that if concrete is produced using RCFA then it would be difficult to control the effect of water-cement (W/C) ratio because of the inaccuracy of measuring W/C. Hansen also added that the high water demand for RCFA will lower strength and (probably) the durability of hardened concrete. He therefore suggested pre-soaking the recycled aggregate before using it to maintain uniform quality during concrete production, but no further details on soaking time in his report.

2.1.1.6 Sulfate Soundness

Recycled aggregate are known to have a higher porosity and more permeability comparing to that of natural aggregate. Concrete durability are known to be negatively affected due to expansions that result from factors such as freezing and thawing actions, alkali-aggregate reactions, sulfate attack, corrosion of the reinforcement, etc. Such expansions depend, to a large extent, upon ingress of water, gases, and aggressive chemicals into the concrete; which, in turn, depend upon permeability. A durable concrete should have low permeability. Thus and permeability affects can be related to concrete durability.

ASTM C33 [7] limits the loss in weight when aggregate (natural sources) is subjected to five cycles of alternative soaking and drying in sulfate solution. When magnesium sulfate (MgSO4) is used the limit for coarse aggregate is 18%, and for fine aggregate is 15%. These limits change to 12% and 10% correspondingly for sodium sulfate (NaS) solutions. Hansen [6] indicated contradictions in some reports concerning the durability of RCA tested according to sulfate soundness. He mentioned that B.C.S.J [15] found NaS soundness losses ranging from 18.4% to 58.9% for RCCA derived from 15 original concretes of different strength.

For fine recycled aggregates of the same sources the values were between 7.4% and 20.8%. These results were confirmed by Kaga *et al* [26] who concluded that most recycled aggregate would be less durable than original aggregates and that the RCA would fail to meet ASTM C33 requirements to a sodium sulfate soundness of not more than 12% loss for coarse aggregates.

In contrast to this, Fergus [27] reported that MgSO4 soundness loss ranged from 0.9% to 2.0% for RCCA and from 6.8% to 8.8% for RCFA. These recycled aggregates derived from different road concrete pavement, which its natural aggregate MgSO4 loss was 3.9% and 7.1% for coarse and fine aggregate respectively. Therefore Fergus [27] concluded that RCCA behaves more effectively than natural coarse aggregates and thus could be more durable. The same applies to RCFA. Due to conflict between American reports [27], which found the durability of recycled concrete generally is more adequate than for natural aggregates, and the Japanese reports [15], which say that the opposite is true, Hansen [6] recommended that additional studies should be made to compare the durability characteristics of recycled and natural aggregates.

2.1.2 Strength of Recycled Concrete Aggregate

2.1.2.1 Aggregate Impact Value (AIV) and 10% Fines Value (TFV)

British standards (BS 882, 1992 [28]) recommend the maximum value for AIV of aggregates as follows; for heavy duty floor 25%, concrete for wearing surface 30% and for other usage of concrete 45%. For TFV British standards (BS 882, 1992 [28]) recommend the following values; for heavy duty floor 150 kN, concrete for wearing surface 100 kN and for other usage 50 kN.

It was reported [6] that Hansen and Narud [12] found the AIV for RCCA (16-32 mm) produced from high strength concrete to be around 20.4%.

Hasaba [19] found that the AIV for 25-5 mm RCCA derived from high strength concrete to be around 23% and the TFV was about 130 kN. For the same size of RCCA, but derived from low strength concrete, the AIV was 24.6% and the TFV was 111 kN. Furthermore Mulheron and O'Mahony [20] reported that TFV for RCCA is significantly lower than that of the Thames Valley gravel, but still exceeds 100 kN. The TFV for recycled debris aggregate (derived from demolition waste includes both concrete and bricks waste) was reported to be around 80 kN – this result was described as not encouraging but that the use of such recycled coarse debris aggregate in the production of anything other than low strength concrete would result in the strength of the concrete being limited by the strength of aggregate.

Hansen [6] concluded RCA derived from all but poorest quality concrete can be expected to pass BS requirements for AIV and TFV (as well as the ASTM Los Angeles Abrasion Loss percentage) for production of concrete wearing surfaces but probably not for granolithic floor finishes. Clearly these reports indicated that the strength of RCA depends on the quality of parent concrete where it was derived from; i.e. high strength and quality of parent concrete produces RCA with good properties. In this research hollow core slab units were used to produce RCA, the strength of these RCA is relatively high, this will be discussed in chapter 4.

BRE [55] carried out an extensive laboratory research on concrete made with natural crushed rock aggregates obtained from 24 different quarries all over the UK. They have tried to study the relationship between aggregate properties to concrete performance. Aggregates properties were investigated in their work and then 550 different concrete mixes were made using those different aggregates. They came to a conclusion that the differences in the performance of concrete could not be related to any single characteristics of the aggregates as shown in Figure 2-5. They have also concluded that no relationship can be established between the amount of free water required (and thus workability) with grading, shape and texture of different crushed rock aggregates, their findings were reported in a graph shown in Figure 2-6.

Padmini, et al [29] studied the Influence of parent concrete on the properties of recycled aggregate concrete. He studied the properties of recycled aggregates derived from parent concrete (PC) of three strengths, each of them made with three maximum sizes of aggregates. Using these nine recycled aggregates, three strengths of recycled aggregate concrete (RAC) were made and studied. He concluded that the water absorption of recycled aggregate increases with an increase in strength of parent concrete from which the recycled aggregate is derived, while it decreases with an increase in maximum size of aggregate. He explained that although the resistance of recycled aggregate to mechanical actions is lower than fresh crushed granite aggregate, the values are generally within acceptable limits and for achieving a design compressive strength, recycled aggregate concrete requires lower water–cement ratio and higher cement content to be maintained as compared to concrete with fresh granite aggregate. He added that for a given target mean strength, the achieved strength increases with an increase in maximum size of recycled aggregate used. For a given compressive strength of concrete he found the split tensile and flexural strengths are lower for RAC than parent concrete, and the modulus of elasticity of RAC is lower than that of parent concrete.

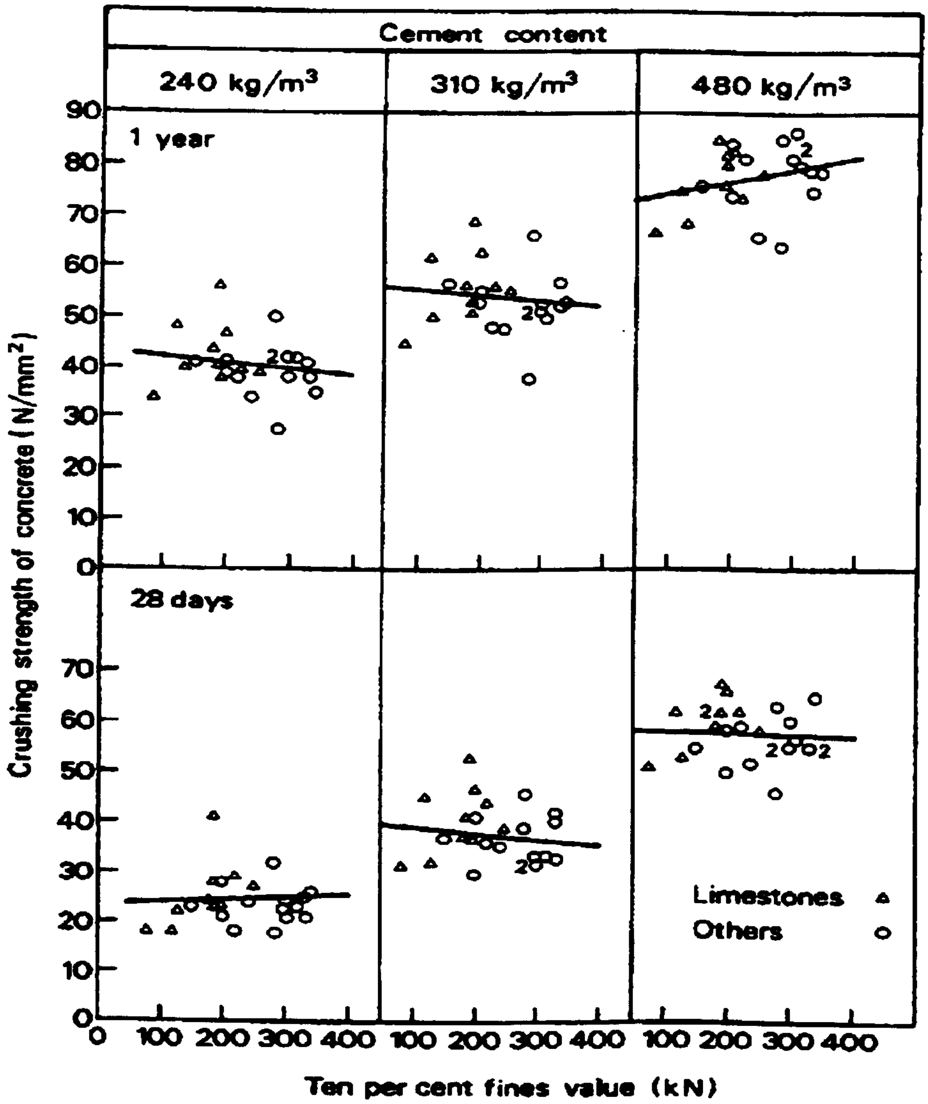


Figure 2-5: Aggregates TPF against concrete compressive strength; BRE [55]

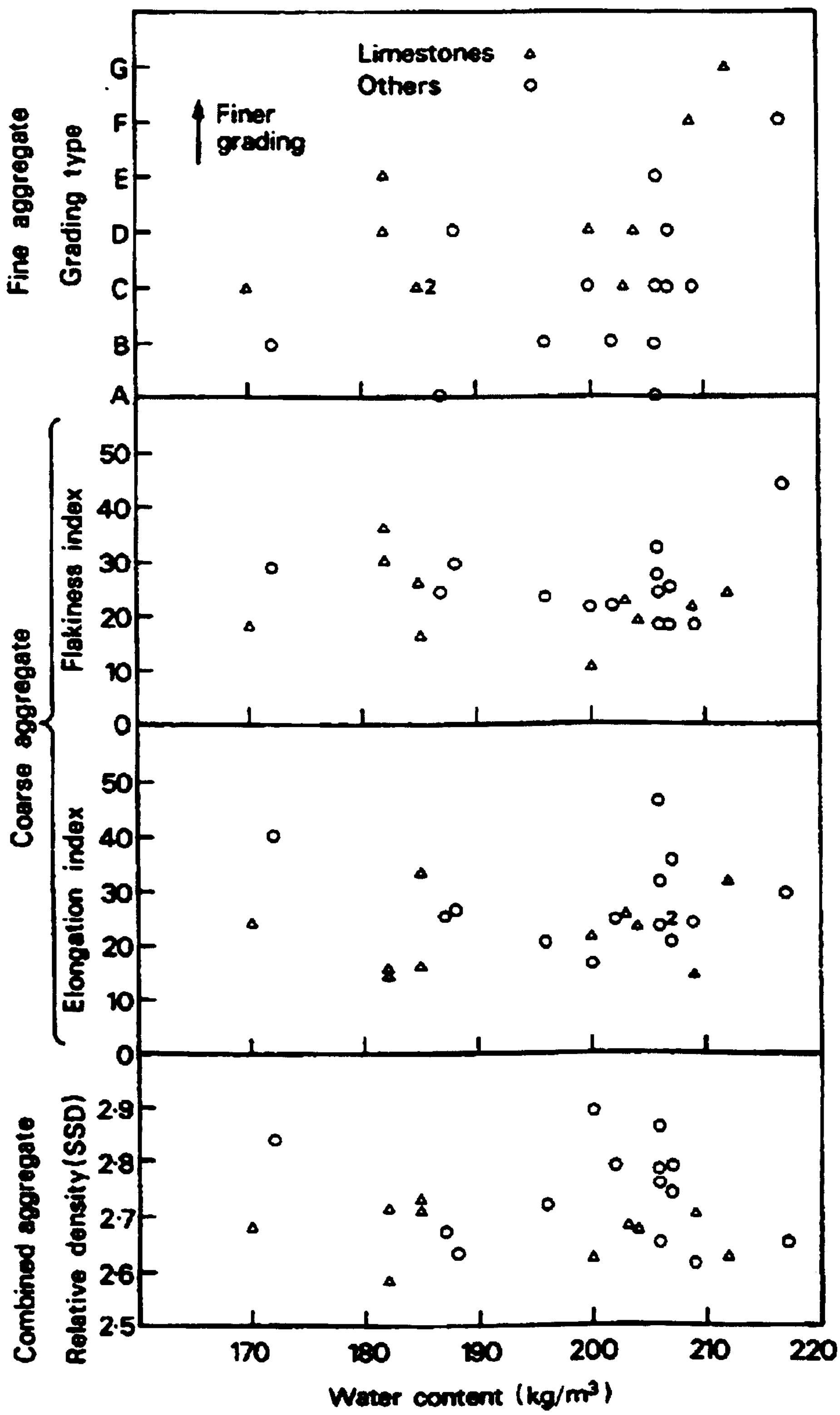


Figure 2-6: Aggregates characteristic and water requirements from BRE [55]

2.2 Concrete with Recycled Concrete Aggregate

2.2.1 Properties of Fresh Concrete with Recycled Aggregate

2.2.1.1 Workability

Hansen [6] reported that Mukai [30] found the concrete with RCCA and natural sand needs about 5% more free water than conventional concrete, while this figure could rise up to 15% if both RCCA and RCFA were used. This was supported by Hansen and Narud [12] and also Ravindrarajah and Tam [31] who have similar findings. On the other hand Rasheeduzzafar and Khan [32] reported that the workability of concrete made with RCA and beach sand is improved compared to concrete with crushed limestone and beach sand. No further explanation was given on how it was improved; however they reported that that the workability severely reduces if both RCCA and RCFA are used.

Similar findings were reported by Mulheron and O'Mahony [20], who explained that concrete with RCCA produced slightly harsher and less workable mixes than the conventional concrete with river gravel coarse aggregate. This was verified by lower values of slump (20 mm for recycled concrete, 100 mm for gravel concrete) compacting factor (0.89 for recycled concrete, 0.93 for gravel concrete) and higher vebe times (4.0s for recycled concrete, 3.0s for gravel concrete). They related this to the particle shape and texture of the RCCA which was more angular and rougher than the gravel aggregate which in turn results in an increased amount of inter-particle interaction and locking.

However for recycled debris aggregate (which is derived from demolition waste includes both concrete and bricks waste) they reported a similar workability compared to conventional concrete. They concluded that the shape and texture of aggregate particles to a large extent influences the workability of fresh concrete.

On a large project where 80000m^3 of RCA was used for production of new concrete Hansen [6] recommended that to avoid a rapid slump loss and early setting of the cement on the fresh concrete made with RCA it would be necessary to pre-soak the RCA prior to mixing, this could be done by immersing the aggregate in water for a sufficient time about one hour. He reported that it takes 15 minutes to saturate 4-28 mm RCCA with water absorption of 5%, 5 to 10 minutes to saturate 0-4 mm RCFA with water absorption of 10% to 17%.

He also explained that pre-soaking the RCA may not influence the compressive strength as there is no significant difference in compressive strength whether the concrete is produced with RCA in air dry or saturated surface dry conditions, providing that the two concretes have the same free water-cement ratio allowing for full absorption of the RCA. Hansen conclusions was based on Hansen and Narud [12] who all found that concrete produced with dry RCCA has lower workability and hardens faster than concrete made with wet RCA, but that compressive strength and modulus of elasticity are almost the same providing they have the same free water-cement ratio. However, Hansen explained that there is much confusion about this issue in the literature as well as in practice due to difficulties in establishing if the aggregates achieved the saturated condition during the mix.

Poon, et al [33] carried out an experimental study on the properties of fresh concrete prepared with recycled aggregates. Concrete mixes with a target compressive strength of 35 MPa are prepared with the use of recycled aggregates at the levels from 0 to 100% of the total coarse aggregate. The influence of recycled aggregate on the slump, bleeding, the effect of delaying the starting time of bleeding tests and the effect of using fly ash on the bleeding of concrete were investigated. The natural aggregates were crushed granite sourced from a local quarry, with nominal sizes of 10 and 20 mm while the recycled aggregates were derived from unwashed construction and demolition wastes. Only coarse recycled aggregates with the maximum nominal sizes of 10 mm and 20 mm were used in their study. They concluded that the use of recycled aggregates at an air-dried state in concrete resulted in higher initial slumps, which took longer to decrease to zero when compared with the concrete prepared with natural aggregates and the use of recycled aggregates also resulted in a higher rate of bleeding and bleeding capacity. They also found that replacement of cement by 25% of fly ash increased the slump of concrete mixtures with and without recycled aggregates and had beneficial effects in reducing the bleeding rate and bleeding capacity, with only minimal negative effects on concrete strength at or before 28 days, but positive effects on the strength at 90 days

2.2.2 Properties of Hardened Concrete with RCA

2.2.2.1 Compressive Strength of Concrete with RCA

The effect on the compressive strength of concrete made with variable percentage replacements of RCCA, RCFA and a combination of both have similar findings according to Hansen [6], Nixon [34] and B.C.S.J [15]. The strength of concrete with RCCA and natural fines is lower to that of conventional concrete with natural aggregate.

Only conclusions referred to by Hansen who reported that it is about up to 20% lower according to Nixon [34] and 14% to 32% lower according to B.C.S.J [15].

In contrast, in another investigation Hansen and Narud [12] used three different types of RCCA derived from high (HC), medium (MC) and low strength concrete (LC). Different target compressive strengths of concrete were cast using these RCCA with nine different combinations. To control the test the mix proportions for the new concrete were similar to that of the original concrete, which was later crushed to provide the RCCA. They found that the strength of RCCA concrete derived from HC grade is almost the same (and in some cases is higher) than the conventional concrete, while it is lower if LC grade RCCA is used. They concluded that for the same water-cement ratio, concrete made with RCCA could have higher compressive strength than the conventional concrete. This depends on the strength of 'original' concrete from where the RCCA is derived.

If the water-cement ratio of the original concrete is the same as or lower than that of concrete made with these RCCA, then the strength of the concrete made with RCCA can be as good as or higher than the strength of the original concrete. However Trevorrow [35] concluded that there is no obvious relationship between the strength of the concrete made with the recycled aggregates and the strength of the original concretes. He performed tests on concrete made using both RCCA and RCFA with different proportions obtained form two laboratories made source concrete of different strength. He added that on the basis of equal workability and with concrete made from RCCA only those made from the weaker source concrete were stronger and the reverse was true when the RCFA was used.

Hansen [6] explains (see Table 2-2) that BSCJ [15] obtained somewhat similar results using RCCA and natural sand. According to Hansen [6] this was confirmed by Yoda *et al* [36] who found an 8.5% increase for concrete with RCCA compared to conventional concrete with same free water-cement ratio.

Table 2-2: Compressive strength (MPa) of original concretes and recycled aggregate concretes made from the same original concretes using recycled coarse aggregate and various proportions of recycled fine aggregate and natural sand [12]

W/C	Standard concrete	RCCA + 100% natural sand	RCCA + 50% RCFA + 50% natural sand	RCCA + 100% RCFA
0.45	37.5	37.0	34.0	30.0
0.55	28.9	28.5	25.0	21.5
0.68	22.0	21.0	17.5	13.0

Kawai *et al* [37] explained that it is possible to produce recycled aggregate concrete with RCCA and natural sand with the same strength as conventional concrete having the same water-cement ratio.

Hansen [6] said, "these observations, indicating a lower compressive strength of concrete with RCCA are explained by Rasheeduzzafar and Khan [32] on the basis of photomicrographs of fracture patterns of recycled aggregate concrete". They explained that when the strength of the control concrete made with conventional aggregate was greater than the strength of the original concrete, the strength of the new mortar and the new mortar-aggregate bond in recycled aggregate concrete is higher than the strength of the recycled aggregate itself or the bond between the old mortar and the original aggregate. This makes the recycled aggregate itself the weakest and, therefore, the strength-controlling link of the composite system. On the other hand, when the control concrete was less than the strength of the old concrete, the inferior quality of the new mortar in the recycled aggregate concrete or its bond with the RCCA forms the weakest link.

Mulheron and O'Mahony [20] reported that at a free water-cement (w/c) ratio of 0.45 the compressive strength for concrete with RCA (from clean graded concrete) is about 59.5 MPa where for concrete with recycled coarse debris aggregate (derived from demolition waste) to be 47.1 MPa, and that for concrete with natural gravel aggregates is 61.5 MPa. They reported the same trend for w/c = 0.54, finding that the compressive strength for concrete with RCCA is about 46.4 MPa, where for concrete with recycled coarse debris aggregate it is 46.0 MPa and that for concrete with natural gravel aggregate is 51.5 MPa. In a different approach, where RCFA was mixed with RCCA, the same test in Hansen and Narud [12] (summarized in this section) was repeated by Hansen and Marga [22] who concluded that the use of both RCCA and RCFA could reduce the compressive strength by approximately 30% compared to concrete with natural coarse and fine aggregate.

They also found that the RCFA always has a deteriorating effect on the compressive strength of recycled aggregate concrete.

This was supported by Ravindrarajah and Tam [31] who explained that the detrimental effect of using RCFA in concrete could be reduced by a partial replacement with natural sand. Hansen [1] also outlined that the compressive strength of concrete made with RCCA and a blend of 50% of RCFA and 50% of natural sand was 10% to 20% lower than the conventional concrete. This could reach to 20% to 40% lower if 100% RCFA were used.

He concluded that to avoid a deteriorating effect of using RCFA in concrete, for instance a reduction in compressive strength and reduction in freeze/thaw resistance, it is recommended to screen out and dispose all RCFA material finer that 2 mm or even to avoid using RCFA below 4 - 5 mm altogether.

He supported this by the fact that the Michigan Department of Transportation [6] had limited the allowable amount of RCFA to 30% of natural sand on Interstate Highway rehabilitation projects and planned completely to prohibit the use of recycled fines on some future work. This is because some tests showed unsatisfactory strength while using RCFA. No further details are reported.

For RCA from precast concrete Collins [45] concluded that the compressive strength for concrete with RCA derived from precast concrete did not show any significant difference to the controls. They have done over 600 concrete specimens, referred to as "small tests".

2.2.2.2 Tensile and Flexure Strength

Investigations reveal that there are small difference in the tensile and flexure strength between concrete with recycled and that with natural aggregate. Hansen [6] indicated that B.C.S.J [15] and Ravindrarajah and Tam [31] reported that there is no significant difference in tensile splitting strength between concrete with RCCA and that with conventional concrete, but it could be reduced by 20% if both RCCA and RCFA were used. Coquillat [38] reported that there is no significant difference compared to conventional concrete even if both RCCA and RCFA were used.

Gerardu and Hendriks [39] found 10% reductions in tensile splitting strength for concrete with RCCA and 20% lower tensile splitting strength for concrete with both RCCA and RCFA. For flexure strength similar findings were reported according to Hansen [6] who explained that B.C.S.J [7] found the flexural strength of concrete with RCA to be between 1/5 to 1/8 of its compressive strength in comparison with conventional concrete.

Malhotra [40], Karaa [41] and Ikeda *etal* [42] also reported a lower flexure strength for RCA concrete. Karaa [41] reported the flexure strength for concrete with both RCCA and RCFA to be 26% lower than the conventional concrete. Hansen [12] refers the large differences reported by the researchers to the differences in quality of the recycled aggregate. Kawamura and Torii [43] reported that the flexure fatigue of concrete made with RCCA was higher than when using conventional concrete. A visual inspection of the fracture surfaces showed that the failure in RCA concrete occurred in the cement mortar portion of the RCA grain, while in conventional concrete occurred in the bond between cement mortar and natural aggregate grains. Therefore he concluded that the higher flexure fatigue strength in RCA concrete is due to the strong bond between cement mortar matrix and recycled aggregate particles.

In another research Guineaa, et al [44] investigated the influence of the interface on the macroscopic fracture parameters of concrete. Eleven concrete batches were cast with the same matrix. Different aggregates, crushed or rounded, from the same quarry were used, and several surface treatments were applied to improve or degrade the bond between the matrix and the particles. Fracture tests three-point bending tests and Brazilian splitting tests (tensile splitting strength) were carried out to determine the fracture energy. The modulus of elasticity and the compressive strength were obtained from uniaxial compression tests. They have concluded that the compressive strength and the modulus of elasticity are strongly affected by the quality of the interface, resulting in a sensible reduction (up to 70% for fc, and 50% for Ec) when the bond is poor; they also found that similar trend applied to tensile strength and added that the use of an adherent matrix can improve the tensile strength well over the matrix strength. They added that the strength of the interface affects the fracture energy (GF) in different ways depending on the shape of the particles; where concretes with crushed aggregates show a higher value of fracture energy (GF).

For RCA from precast concrete the DETR report [45] concluded that the flexural strength for concrete with RCA derived from precast concrete did not show any significant difference to the controls. They also produced full-scale standard hollow core slabs using 20% RCCA and 20% RCCA+10% RCFA. Some of the slabs were set under load up to about one year and they found that very slight increases in deflection which are of no significance in commercial use and are within normal range of variation for the testing of identical samples from the controls.

An interesting research carried out by Jianzhuang [46] who investigated the bond behaviour between recycled aggregate concrete and steel re-bars and tried to establish a bond stress versus slip relationship between recycled aggregate concrete and steel re-bars. Three different RCA replacement percentages were used in the study 0, 50% and 100%, respectively. Normal concrete served as reference concrete. The water/cement ratio was kept constantly to 0.43 but it should be noted that in their investigation the RCAs were pre-soaked by additional water before mixing and the amount of this additional water was calculated on the basis of the saturated surface-dry condition.

Using a pullout in accordance with a Chinese standard (GB50152-92) [46] they have concluded that the bond strength between the recycled aggregate concrete and steel re-bars is higher than the one between normal concrete and steel re-bars and they stated that the anchorage length of steel re-bars embedded in the recycled aggregate concrete with 100% RCA can be chosen as the same for normal concrete under the condition of the same compressive strength of concrete. They also found that the general shape of the load versus slip curve between recycled aggregate concrete and steel re-bars is similar to the one for normal concrete and steel re-bars. They explained their findings on the possibility that the values of modulus of elasticity of the recycled coarse aggregate and the cement paste of the recycled aggregate concrete might be similar but no further explanation was given.

Another research carried out by Choi and Kang [47] to investigate the bond performance between concrete with recycled aggregate (RAC) and reinforcing bar and also investigated the shearing strength and shearing failure of concrete. They have used w/c ratios of 0.4 and 0.50 and three replacement ratios of recycled aggregates (30%, 50% and 100%). The specimens for the pull-out test were three 150 x 150 x 150 mm cubes. 16 HD (high-deformed bar) high-strength bars with yield strength of 800 MPa were used to prevent the rebar from yielding before bond failure occurs on the attaching surface between the rebar and the concrete. He also found that for a w/c ratio of 0.4, and up to a replacement ratio of 50%, the bond stress–slip relationship for RAC shows a tendency similar to that of normal concrete, regardless of the quality of RA and the replacement ratio. However, for a w/c ratio of 0.5, up to a replacement ratio of 50%, the bond stress–slip relationship reacts sensitively to the quality of the RA and the replacement ratio, and appears to be better than the bond stress–slip relationship for normal concrete. The shearing stress–shearing strain relationship for RAC is largely affected by the grade and the replacement ratio of the RA. The shear stiffness of RAC decreases as the replacement ratio of RA increases. The bond strengths of RA are higher than those of normal concrete. For a w/c ratio of 0.4, the bond strength with concrete of high quality grade of recycled coarse aggregate is similar to or higher than that of normal concrete, but the bond strengths recycled coarse aggregate. For a w/c ratio of 0.5 the bond strength of RAC is not greatly influenced by the quality and the replacement ratio of the RAs.

2.3 Durability of Concrete with RCA

2.3.1 Water absorption and Permeability

Trevorrow [35] reported that there is a significant increase in both the porosity and permeability in concretes made using recycled concretes aggregates compared with the control made from all natural materials. He added that there is a good correlation between the permeability of the source concretes and the permeability of new concretes made using the coarse fraction of the recycled material from these source concretes. Hansen [6] also mentions that according to Rasheeduzzafar and Khan [32] the permeability of RCA concrete could be related to the strength of the original concrete where the recycled aggregate was derived from. Their conclusion was based on a comparison of water absorption of concrete with RCCA made with different W/C ratios (therefore different compressive strength) with that of conventional concrete made with identical variance of W/C ratios. If the compressive strength for the original concrete, from which the recycled aggregate was derived, is lower than the compressive strength of both concretes with recycled and conventional aggregate, there could be no significant difference in the water absorption of with RCA versus conventional concrete.

The situation is completely different if the compressive strength of both concrete with RCCA and conventional concrete is lower than the compressive strength of the concrete from which the recycled aggregate is derived. In this case they found the water absorption of concrete with RCCA could increase to three times compared to corresponding conventional concrete.

This was explained by Hansen [12] who said such RCCA would contain a large volume fraction of more porous RCCA, which is distributed in a relatively dense matrix, while the conventional concrete contains natural aggregate and comparatively dense natural aggregate in the same relatively dense matrix.

2.3.2 Freeze/Thaw Resistance

Hansen [6] reported that Malhotra [40], Buck [48] and Nixon [34] found that the Freeze/thaw resistance for concrete with RCCA is not significantly lower than that of conventional concrete, and is sometimes higher. He also supported this with B.C.S.J [15] who reported that there is no significant difference in freezing and thawing resistance for concrete with RCCA and that with conventional concrete. There is a significant difference if both RCCA and RCFA were used. This was supported by Hasaba *et al* [19] who reported that conventional concrete has better Freeze/thaw resistance than concrete with either RCCA alone, or both RCCA and RCFA; No reasons were reported.

Mulheron and O'Mahony [20] measured the ultrasonic pulse velocity in concrete as a function of the number of freeze and thaw cycles, see Figure 2-7. The substantial difference in behaviour between the specimens immersed in plain water and those immersed in saturated sodium chloride solution is explained by the presence of de-icing salts increasing the rate and extent of attack on concrete exposed to alternate freezing and thawing. They found that all of the concretes immersed in saturated sodium chloride solution show some damage after 5 cycles and the first one to fail completely was the control mix at only 20 cycles.

The same behaviour was reported for plain water solution. It was concluded [20] that concrete made with RCCA showed better resistance to the effects of freeze-thaw conditions than Thames Valley gravel. After 42 cycles, the concrete with RCCA shows no change in pulse velocity while for the control mix the reduction was about 18%. This may also enhanced the durability in which that the concrete with RCCA have more porosity than conventional concrete and this could provide a sufficient number of air filled macro-pores which in turn could reduce the pressure resulting from ice formation. The gravel aggregate showed better resistance to freeze-thaw condition than RCCA, see Figure 2-8.

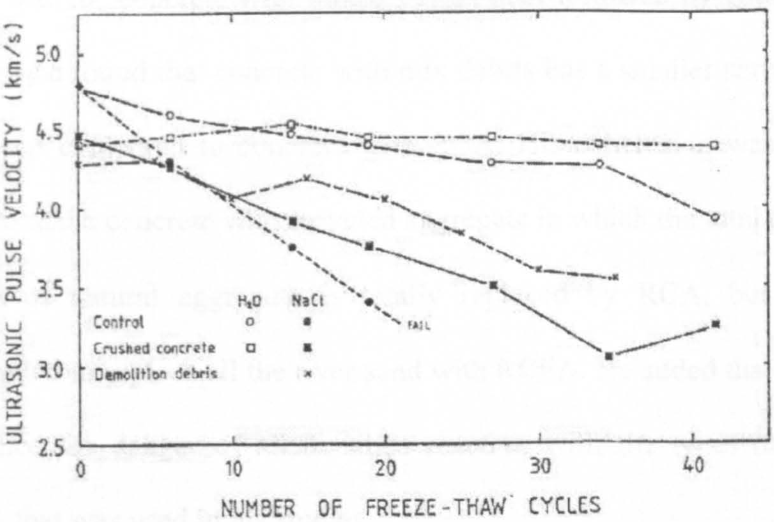


Figure 2-7 Ultrasonic pulse velocity of hardened concrete specimens subjected to alternate freezing and thawing (from Mulheron and O'Mahony(20)).

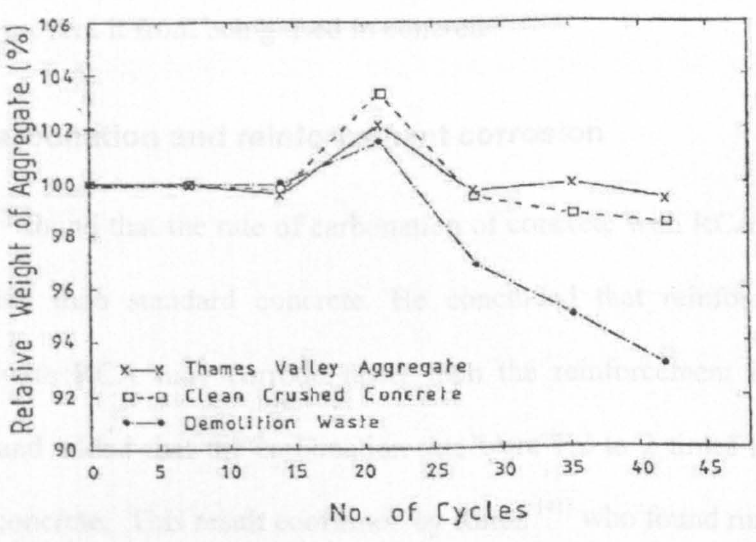


Figure 2-8 Relative weight loss of aggregates as a function of freeze-thaw cycles (from Mulheron and O'Mahony(20)).

2.3.3 Chloride Ingress, Shrinkage and Creep

Fraaij [49] studied the chloride penetration in concrete with RCA and concrete with mixed debris (includes bricks and concrete debris) and then compared it to standard concrete. He found that in all cases the intrusion was limited to about 20 mm where the lowest was for standard concrete and the highest intrusion was for concrete with mixed debris then followed by concrete with RCA. Also he found that concrete with mix debris has a smaller shrinkage and higher creep compared to concrete with RCA. His conclusion was that it is possible to make concrete with recycled aggregate in which the total fraction of 4-32 mm of natural aggregate is totally replaced by RCA, but it is not recommended to replace all the river sand with RCFA. He added that there was no evidence for danger of alkali silica reaction with the types of recycled aggregate that was used in his studies.

Research by Fujii [50] showed that concrete with RCA has a higher shrinkage value by 20% - 30% than standard concrete, but that this difference is not crucial to prevent it from being used in concrete.

2.3.4 Carbonation and reinforcement corrosion

B.C.S.J [15] found that the rate of carbonation of concrete with RCA was about 65% higher than standard concrete. He concluded that reinforcements in concrete with RCA may corrode faster than the reinforcement in standard concrete and added that the carbonation rate were 1.2 to 2 times higher than standard concrete. This result confirmed by Karaa [41] who found rust after two months on reinforcement bars with RCA concrete.

3 Research Background

Introduction

This chapter shows the result of an investigation carried out on the concrete industry to clarify and to prioritise the obstacles they face using recycled concrete aggregate in new concrete. It is decided that to investigate the effects of using RCA may not be comprehensive without studying the process of producing these RCA. A questionnaire was sent for this purpose and its outcome is discussed. Among other key issues that were raised in the questionnaire, we believed crushing methods and their effects on recycled aggregate was a key factor. From this point it is decided for this research work to produce recycled concrete aggregate (RCA) by using three (common) types of crushers. The crushers are Jaw, Cone and Impact. An explanation of these crushing machines is simplified in this chapter. A comparison of these crushers was achieved by carrying out a detailed investigation on their output of RCA.

To achieve a fair comparison, a great effort was made to ensure the input for the three crushers was constant. The constant input is a proprietary precast concrete hollow core slab units (produced by Richard Lees Ltd. UK now Tarmac Top Floor). The objectives were to promote the usage of high quality recycled aggregated derived from rejected hollow core slab units and to highlight the crusher(s) that have a better performance for production of the RCA through examining the different characteristics of the RCA considering the three different ways of crushing. The RCA were compared with each other as well as with natural crushed limestone aggregates according to the current British Standard Specifications (BS) and British European Standards Specifications (BS EN). The natural limestone is the crushed carboniferous limestone obtained from Tarmac Quarry Products, Retford.

3.1 Interests of Concrete Industries Organizations

It could be seen from the literature review that several investigations have been carried out on concrete made with RCA. A major concern is the strength of hardened concrete made with coarse RCA, called RCCA, while other aspects seems to come next. It was decided that it would be logical to try to focus this research on certain aspects linked to the strength of concrete. Doing so will help to enhance the confidence in (more) usage of RCA which may lead to improvement in efficiency and appropriate to use safely with compliance to current British Standards (BS) at the time of the laboratory work described in this thesis. Compliance with BS appears to be the main interest of many different firms in the precast concrete industry that has the enthusiasm in using this RCA but reluctant to do so as there are no clear standards to follow.

In an attempt to clarify the obstacles in handling and using RCA for new concrete production, and then to acknowledge and to focus on the priority for such area(s), a questionnaire designed by us (see Appendix 1) was sent to different firms and academic institutions in the UK and some European countries.

The input of the questionnaire was selected based on criteria related to recycled aggregate production, which in consequence have effects on concrete properties both fresh and hardened. The factors were carefully selected after several meetings with related academic and industry personals.

The number of returns from 100 questionnaires was 65 and the results are summarized in Table 3-1. It was seen that the outcomes of the questionnaire are quite variable, indicating that the interest of the concrete industry is inconsistent. As expected, strength appears to be the main priority. Impurities come next. However, this should not be a critical problem in disposed precast concrete since the only concern there is that steel rebar and concrete could be easily separated during the crushing operation.

There is more concern on the durability of concrete with RCA than the workability according to the questionnaire, while a moderate interest in water absorption was indicated. This research intended to consider these properties since RCA has a greater water absorption capacity than natural aggregate, which could affect the concrete performance in general.

Lower interest on both crushing methods and mixing design were indicated, however some previous research work has recommended the need to study the crushing methods [1] and for completion we believed this research is ought to highlight and study the production of RCA. The crushing methods could have an influence on the properties of RCA.

For example, quality, shape and texture, angularity and flakiness, and such properties might influence the workability, degree of compaction and the durability of concrete made with these RCA.

The shape and grading could be included within the crushing method. Similarly adapting and controlling the mixing design could leads to improvement to the performance of concrete with RCA.

Although reasonable interest on the bond strength was indicated, but it is an essential area to investigate for either the bond between recycled aggregate and mortar matrix and the bond between the reinforcement and concrete made with RCA. Some interest in admixture was indicated where it could be employed to reduce the water need and to improve the workability.

There was a low interest in latent hydration although this could be very important in reusing freshly disposed concrete in a precast plant. Later in this research, it is seen that many factors from the outcome of this questionnaire have been logically considered.

Table 3-1: Questionnaire Results: Starting from high priority

Suggested Areas	High	Med.	Low
Compression Strength	75%	25%	0%
Impurities of RCA	75%	17%	8%
Durability	67%	17%	17%
Tensile Strength	58%	42%	0%
Admixtures	58%	33%	9%
Alkalis - aggregate reaction	58%	17%	25%
Grading of RCA	50%	50%	0%
Bond Strength (RCA with new Cement Mortar)	50%	42%	8%
Shrinkage	50%	42%	8%
Water Absorption for RCA	42%	50%	8%
Shape of RCA	42%	50%	8%
Crushing Methods	42%	42%	16%
Cracking	42%	42%	16%
Compaction	33%	58%	9%
Static Modulus of Elasticity	33%	58%	9%
Workability	33%	42%	25%
Segregation	25%	67%	8%
Creep	25%	50%	25%
Mixing Design	25%	42%	33%
Water bleeding	17%	58%	25%
Texture of RCA	17%	58%	25%
Dynamic Modulus of Elasticity	17%	58%	25%
Latent Hydration	9%	58%	33%
Poisson's Ratio	0%	58%	42%

3.2 Production of recycled concrete aggregate

Crushing methods were considered to be one of the initial criteria requiring investigation, since it has a fair influence on the properties of the aggregates and subsequently on the performance of concrete. It is the basis for the production of the RCA and this gives more concern towards it.

3.2.1 Origin of recycled concrete aggregate

Waste material from hcu, see Figure 3-2, is high grade and uncontaminated material. The parent concrete is hard with 28-day compressive cube strength between 50 and 80 N/mm^2. It is manufactured from Portland Cement, and from clean and reliable sources of 10 mm to 14 mm limestone or gravel.

The grading of its coarse and fine aggregates is carefully controlled, and together with low water cement ratio, the resulting concrete is of a high density and low porosity. Small quantities of admixtures such as fly ash and Superplasticizer are sometimes used. If procedures were put to reuse such high quality concrete waste as recycled aggregate within the same industry (production of hcu) or other industries it will lead to a greater efficiency via significant reduction in the demand for natural aggregates, reducing pressure on landfill sites and disposal costs. A standard mix proportions for hcu is as follows:

Table 3-2: Mix content for the parent concrete (hcus)

14 mm	***10 mm***	***Sand***	***Cement Class 52.5N***	***Pozzolan***	***Water***
340 kg	***440 kg***	***500 kg***	***200 kg***	***60 kg***	***50 kg***

Since the crushing method was one of the main criteria of this research work, it was decided to limit the parent of the recycled aggregate to one source. This was necessary in order to achieve a fair comparison between the crushers. The source chosen to be the rejected units of hollow core slab, see Figure 3-1.

These units could be rejected because of any of the following reasons: unsatisfactory finishing, failure to meet dimensional tolerances, low cover to reinforcement or structural damage caused during lifting or transportation. However, the strength is usually not a factor.

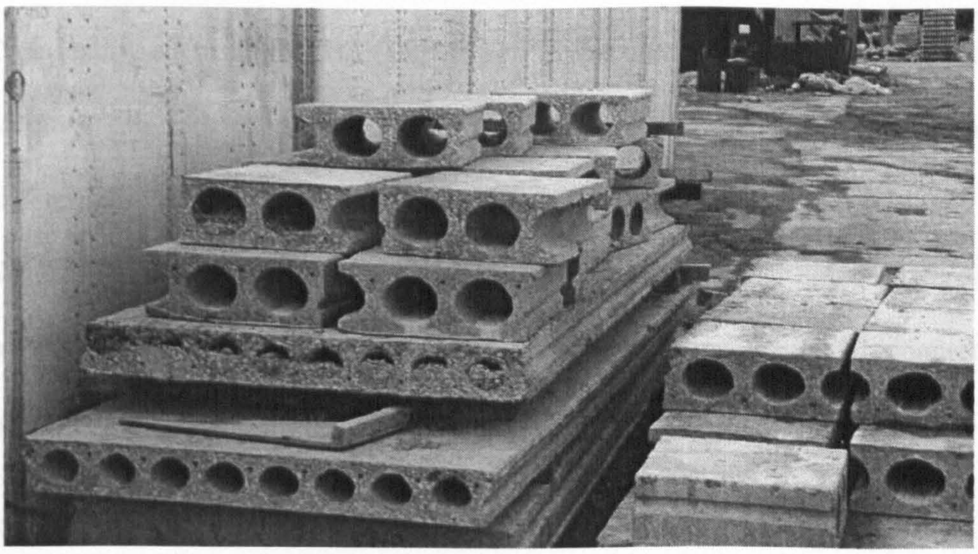


Figure 3-1: Rejected hollow core slab units used for production of RCA using different crushing machines

Figure 3-2: Waste of precast concrete in the precast concrete factory

3.2.2 Strength of rejected units of hollow core slab

Core testing according to BS 6089:1981 [51] was used to measure the strength of those rejected hollow core slab units. Six cores were taken from three different slabs of over a year old and the average strength was found to be 75 N/mm^2. These hollow core slab units were crushed using three different crushers described in the following sections to produce recycled concrete aggregates.

3.2.3 Crushing Machines

In quarrying industries, there is a wide range of information about plant, machinery and the application of this equipment to the requirement of the industry. Description of plants varies greatly regardless the fact that they may produce similar products. This is because there are so many variables involved and what may be found to be suitable for one plant may not apply to another [52]. It is not feasible to distinguish the actual reason why one quarry uses one type of machine while another quarry uses something quite different considering they have the same production operation. Another factor that probably contributes to this confusion is the manufacturer's literature, which provides useful information, even though it is prepared to encourage the increase of the sale of their product.

This part of the research considered the employment of some crushers for the advantage of producing RCA and not the mechanical theory of manufacturing procedure. In the following sections a brief description of the crushing machines that have been utilized in this research are summarized.

The number of crushing machines was limited in this investigation to three types: Jaw crusher, Cone crusher and Impact crusher. These three specific types were chosen due to the fact that they are quite common types in the quarry industries and are being used on production of recycled aggregates, most commonly for roadwork. It should be noted that these crushers are designed to be used in quarrying natural resources and not for recycling purposes.

3.2.3.1 Jaw Crusher

Jaw crushers are one of the most common types of crushers being used in quarries. Its principle could be summarized in that the feed is subjected to repeated pressure as it passes downwards and is progressively reduced in size until it is eventually small enough to pass out of the crushing chamber.

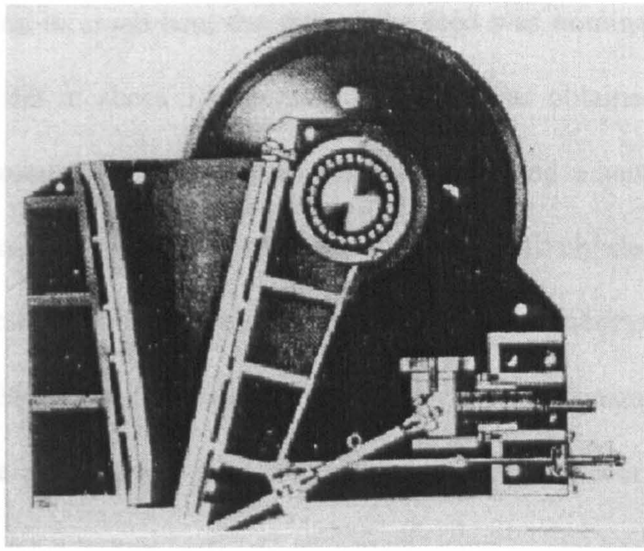


Figure 3-3: Sectional view showing the mechanical details of a single- toggle jaw crusher (from Brown[53])

The pressure is caused by the two jaws, which form the wedge shaped crushing chamber. The size of jaw crusher is usually defined by its feed opening, for example, a width of 1300 mm and a gape of 1100 mm, see Figure 3-3. The size of machine also affects its speed. The rotating speed decreasing as the crusher size increases. The angle between the crushing faces known as the nip angle is normally between 19° and 22°. This angle is a significant reduction when compared with the machines of 20-25 years ago.

This is principally to allow the crushing force to be transmitted to very hard rock without a tendency to the feed rock to rise itself out of the crushing zone and so cause abrasive wear to the liners and restrict capacity [53].

The setting is usually measured as a close side setting (CSS), i.e. when the jaws are at their closest position, and sometimes as open side setting (OSS) with the jaws at their greatest distance apart.

In the operation to crush hcu, the size of the feed was nominally 25mm and the CSS was set at about 15mm. The feed size was obtained by using an electric saw to cut the unit slab into smaller pieces and a hammer was used occasionally as well. The crushing procedure was relatively slow because it is a small university based laboratory jaw crusher. It took 6 hours to produce one ton of RCA. However, the resulting RCA was good in appearance and seems to be slightly elongated with no distinguishable dust on the coarse particles.

3.2.3.2 Cone Crusher

Cone crushers are one of the major categories of Gyrating Crushers, which have developed into being one of the most important types of machine in use in quarrying, according to Mellor[52]. He explained that it uses a repeated compression action with a fixed and a moving crushing member. The moving crushing member, known as the head, is in the form of an erect truncated cone, which revolves within the fixed member, in the form of a frustum of an inverted truncated cone see Figure 3-4. As the lower end of the shafts rotate within an eccentric member, a gyrating effect is created. This means that there is always an area of the crushing chamber where the feed is under pressure and a continuous crushing action is achieved with a fairly constant power load.

The wide displacement of the head at each stroke is at speed that permits the pieces of rock to fall freely by gravitation and be caught further down by the rising head on its return stroke.

Figure 3-5 shows the step by step free fall of the material through the crushing chamber. The long stroke and high speed give agitation to the rock in its passage through the chamber. At the lower extremity of the crushing chamber, the faces of the two crushing members are so shaped that they are parallel for a section. When properly fed the larger pieces are assured of having at least one dimension equal to, or less than, the setting, quoted as closed-side setting (CSS), and certainly all products less than twice the CSS. In the operation to crush hcu, the size of the feed was up to 350 mm and the CSS was set at about 14 mm, electrical saw was used to reduce the feed size.

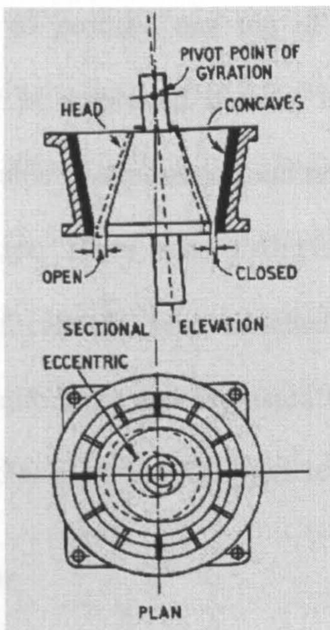


Figure 3-4: Diagram showing the principle of the gyratory cone crusher from Mellor [52]

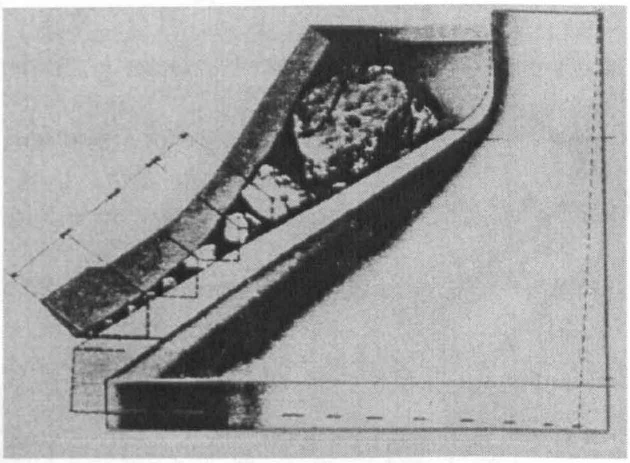


Figure 3-5: The principle of crushing in cone crusher from Mellor [52]

It took almost one hour to produce one ton of RCA using this crusher, however the speed could be improved if there is a continuous feed. The resulting RCA was acceptable in appearance and seems to be slightly towards flaky and elongated in shape. There is no distinguishable dust on the coarse particles. The amount of cement paste attached to the virgin aggregate visually appeared to be quite large and estimated to occupy half the surface area and could be seen higher in some RCA particles.

3.2.3.3 Impact crusher

Impact crushing could be described as impact breaking since the rock is not crushed but fragmented by kinetic energy imported into the feed material. The kinetic energy is introduced by a rotating mass (the rotor), which projects the material against a fixed surface causing it to shatter. The process causes the material to break along its natural cleavage planes and this gives a good product shape free from stress.

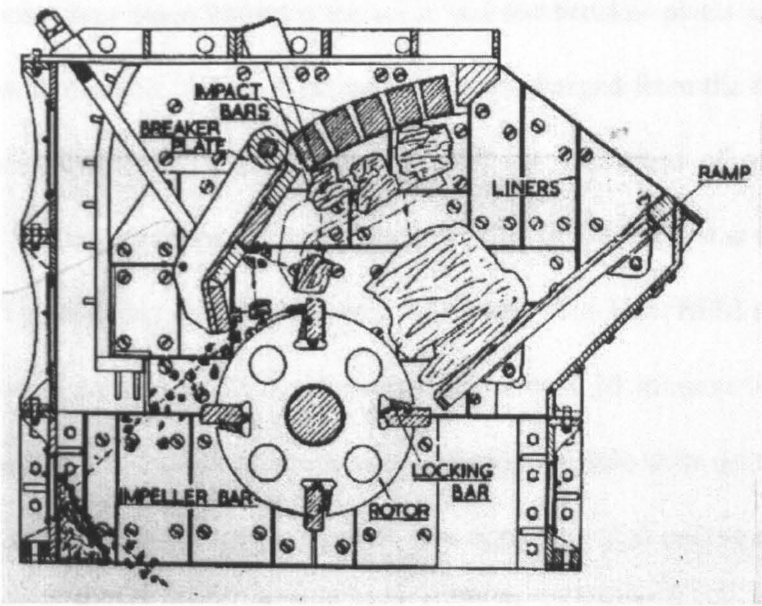


Figure 3-6: Sectional drawing showing the major components and path of stone in a single - rotor impact crusher from (Brown and Mech.[53])

Mellor [52] indicated that there is a formula for the principle of impact crusher: $E = (MV^2)/2$, where E = energy, M = mass and V = velocity. Either a higher mass of the rock or a higher rotor speed would increase the kinetic energy and therefore provide a higher reduction ratio. Generally, the principle of breaking action could be summarized as follows, see Figure 3-6. The feed material is struck by the rotor or hammers which deliverer a high impact load to the material and thereby creating internal stress.

This provides an initial disintegration, and therefore, the design of the crushing cavity has to take into account the expansion of volume required as a solid explodes into numerous fragments. These fragments then strike the impact plates around the crusher body causing further particle size reduction

Some action takes place between the rotor and the breaker plates as stone to stone impact collision occurs. The material is discharged from the crusher by gravity, sometimes via a grid to ensure that the minimum of oversize is produced. In the operation to crush hcu, the size of the feed was nominally 500 mm. The opening size was 700 mm x 500 mm with 1800 RPM rotation. It is very fast in producing RCA where it takes almost 10 minutes to produce one ton of RCA. However, there was a distinguishable dust on the coarse particles. It produces a larger amount of fine aggregate than coarse aggregate. The resulting RCA was less elongated towards rounded shape with acute edges. The amount of cement paste attached to the virgin aggregate also visually appears to be quite large.

3.3 Conclusion

It is clear that the concrete industries are quite variable, so logically their interests in recycled concrete are quite variable as well. However there were some fundamental issues in common between them, for instance the strength of concrete with RCA and the factors affecting it. Durability of concrete and impurities of RCA also were of a high concern on the concrete industry.

The precast concrete hollow core floor industry produces a considerable amount of waste concrete elements. The total waste generated in the UK is around 5% of the production so this high quality waste has been recycled for use as replacement aggregate in the concrete (RCA).

Crushing methods were considered to be the initial criteria requiring investigation in this research work as it is believed that it has a fair influence on the properties of aggregates and subsequently on the performance of concrete. The crushing machine was limited in this investigation to three types: Jaw, Cone and Impact crusher. These were chosen because they are quite common in the quarry industries and recently are being used for production of recycled aggregates, most commonly for RCA used in roadwork.

All three crushers produced acceptable shape of RCA. It was very clear that certain amount of cement paste were adhered to the recycled aggregates that are produced from all three crushers.

It was not possible to distinguish visibly which crusher produces RCA with lesser amounts of adhered mortar in comparable to other crushers. However, Initial appearance revealed that the RCA come out to be relatively the same but slightly better shape could be seen from impact crusher where the particles emerged more rounded and less elongated but with considerable amount of dust on them. This is consistent to that the amount of fines obtained was high for the impact crusher, followed by cone and then jaw crusher. The large amount of dust produced is related to the mechanism of the impact crusher where it has a large reduction factor (from the feed to the output) and consequently it produces larger amount of fine aggregate than coarse aggregate. This agrees with Boesman's findings [54] who reported that impact crushers produce more angular particles than jaw crushers, and they produce twice the amount of fines than jaw crushers for the same maximum size of RCCA. This is of particular importance as other research [55] found that as finer sand are used the water content of concrete of the concrete has to be increased to maintain the workability and also found that when very fine particles passing 75μm are increased they may act as a lubricant and could increased the workability of concrete. Further physical property details and their effects will be discussed in later chapters.

The speed at which the quantity of RCA was produced by the different crushers was convenient for the amount of material required in this research. It is quite high for the impact crusher while the cone and jaw were relatively the same, but slower than the impact.

4 Characteristics of Recycled Concrete Aggregates

Introduction

The quality of aggregate in concrete is of considerable importance. Approximately three-quarter of the volume of concrete is occupied by aggregate. It affects the strength of concrete, as well as the durability and the overall structural performance. The strength of concrete is known to depend primarily on the water to cement ratio and the degree of compaction; where the degree of compaction is influenced greatly by the physical properties of aggregates. This should give concern towards the source and the method of obtaining the aggregates. The properties of crushed aggregate generally depend not only on the nature of the parent material but also on the type of crusher and its reduction factor. This section will study the physical and mechanical properties (strength) of recycled concrete aggregate (RCA) obtained from the three crushers. The results are compared with each other as well as with natural crushed limestone under British Standard Specifications. The natural limestone is the crushed carboniferous limestone obtained from Tarmac Quarry Products, Retford. This comparison aimed to indicate which crusher(s) performs more effectively for the production of RCA.

4.1 Physical Properties

Tests that are the most commonly used in the aggregate industry were used to investigate the physical properties of the RCA obtained from the three crushers.

The tests are flakiness index, which affects the capability of concrete to be easily compacted, as more effort is required to release air from beneath flaky particles, i.e. less air voids means denser mix and better quality concrete. Angularity number is another common test, defined as a measure of self-compacting and a measure of good shape. It is believed that adequate angular aggregates tend to give better interlocking within the matrix, which lead to a higher concrete strength. It is known that the stress at which cracks develops depends largely on the shape of the coarse aggregate; smooth gravel leads to cracking at lower stresses than rough and angular crushed aggregates.

Water absorption and density are related to the presence of both internal and surface pores in the aggregates. Such pores have an effect on the bond between the aggregates and the cement paste through the extent of cement paste ingress in these pores; the better the ingress the better the bond the higher the strength of concrete. Sampling procedures including riffle box was used according to BS EN 932-1:1997 Methods for Sampling.

4.1.1 Flakiness Index

BS 812, Part 105-1, 1989[56] was followed to measure the flakiness index. It is recommended that a particle is considered to be flaky if its thickness is less than 0.6 times the mean sieve size of the size fraction to which it belongs.

Certain types of elongated sieves were provided to measure the flakiness index, which could be calculated as follows:

$$\text{Equation 4-1:} \qquad \text{Flakiness Index} = \frac{M_3}{M_2} x\ 100$$

Where M_3 stands for masses of the aggregates that passes the provided elongated sieve and M_2 for the sample weight before sieving through the elongated sieves. BS 812-103 [8] limits the value of the flakiness to not exceed 40 for crushed aggregate and 50 for natural gravel. This limitation is recommended in order to avoid entrapped water and air lying beneath flaky aggregate since this could lead to a deteriorating effect on the concrete by affecting its workability by causing more voids and lesser-consolidated matrix.

Table 4-1: Values off Flakiness Index & Angularity Number for recycled aggregate derived from different crushers

RCA	**Impact**	**Jaw**	**Cone**	**Natural Limestone***	**BS Limits**
F.I %	9	15	21	7	≤ 40
A.N.	9	6	11	3	--

*Derived using series of jaw crushers

The values given in Table 4-1 are the mean of two tests carried out on 10-14 mm sizes. Using two tests samples considered satisfactory because of consistence results, see Appendix 2 for full data. The data are markedly different for each type of crusher, but are much lower than the BS 882 [28] limits. The impact crusher has the lowest flakiness index, producing about 60% and 40% less flaky recycled aggregate than from the cone and jaw crushers, respectively. Jaw crusher could produce about 30% less flaky recycled aggregate than the cone crusher

Although the cone crusher has the highest flakiness index (21) this is still half of the upper limit (40) of BS 882 [28]. This is considered as sufficient but slightly worrying in relation to acute edges causing a large reduction in the ten-per-cent fines load. In comparison to a research carried out by Teychenne [55] the flakiness index values of hcu RCA (given in Table 4-1) are found to out perform different natural crushed aggregates that obtained from different quarries which covers a wide variety of geographical distribution throughout the UK. Teychenne [55] reported a considerable range in aggregates particle shape as measured by the flakiness index obtained from 24 different quarries in the UK. The values are varied from 10 for Dolomitic Limestone in Durham, 16 for Dolomitic Limestone in Gloucestershire, 36 for Oolitic Limestone in Dorset and 44 for Recrystallised Quartzite in Warwickshire; the average value for the 24 quarries is reported to be around 24, which is higher than values of hcu RCA. However, these values appears to have limited effects on the strength of concrete as will be seen in later chapters.

4.1.2 Angularity Number

British Standard BS 812, Part 1, 1975 [57] defines the concepts of the angularity number (AN) as 67 minus the percentage of solid volume in a vessel filled with aggregates in a standard manner. The higher the number the more angular is the aggregate. The range is usually between 0 to 11 and measured by this equation:

Equation 4-2: $$\text{Angularity Number} = 67 - \frac{100 \times M}{C \times G_A}$$

where M stands for the weight of the sample, C is the weight of the water needed to fill the cylinder and G_A is the aggregate particle density.

The method is not popularly used, but nevertheless provides a useful indication of the ability for aggregates to compact. The size of coarse aggregate used in these tests was 10-14 mm. The results are the mean of two tests.

It was found that the jaw crusher produced less angular RCA than the impact and cone crushers, while there is relatively little difference between the angularity number of RCA produced from the impact and cone crushers. The results given in Table 4-1 indicate that the shape of the RCA produced by the cone and impact crushers was bordering near the end of the acceptable range, i.e. AN = 9 to 11, whilst the natural limestone and jaw crushed RCA was within the desirable range of AN = 3 to 6. According to Kaplan [58] there is an inverse correlation between AN and the compaction factor (CF), a result which is confirmed in Figure 5-3 where the CF for the cone crushed RCA replacements is considerably lower than for all other cases.

Another theory noted is that a higher angularity number, although it might reduce the workability, could lead to a higher strength especially for high strength concrete; see Neville [59] Particle Shape and Texture section. The higher the angular and rougher surfaces are the higher the bond strength between the aggregate and the hydrated cement, i.e. better mechanical interlocking between the aggregate and the cement paste which contributes to the strength of concrete in compression and flexure. Similarly Guineaa [44] have explained that compressive and tensile strengths are greatly affected by the quality of the interface (aggregate to matrix) resulting in a sensible increase when the bond is stronger, as in the case of using more angular crushed aggregates.

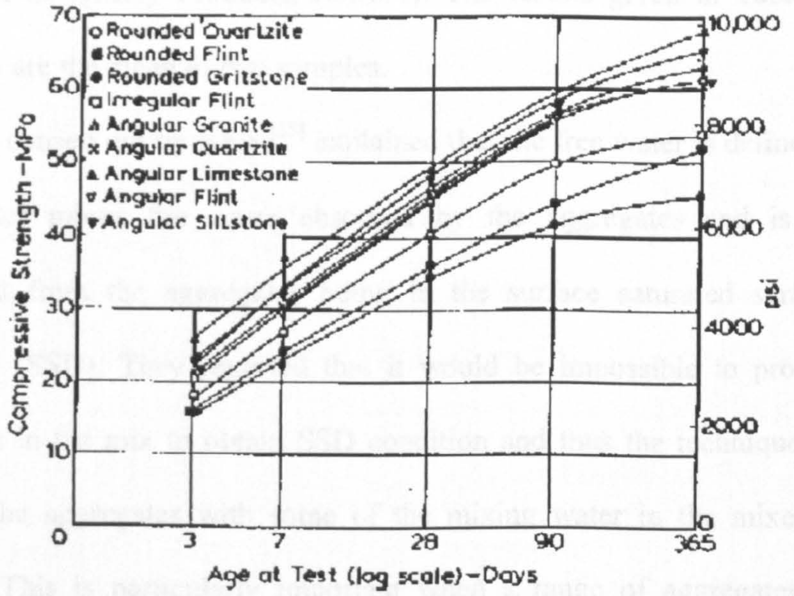


Figure 4-1: Relation between compressive strength and age of concrete made with various aggregates (W/C = 0.5) (Crown copyright) Neville [59]

Considering the fact that the recycled aggregate obtained from high strength concrete showed more angular and rougher surfaces, this may give similar or slightly better performance than the natural aggregate in strength. More about the concrete strength will be discussed in later chapters.

4.1.3 Density and Water Absorption of Recycled Concrete Aggregate

The presence of internal pores in the crushed particles has an influence on the porosity and absorption properties of RCA. These properties have a substantial effect on the workability of concrete with a low water content and low water-cement ratio, especially used in hcu production. They also have an influence on the bond to hydrated cement paste, as well as the resistance to freezing and thawing, and to a lesser extent carbonation. Water absorption and density were measured for the RCA and natural crushed carboniferous limestone (obtained from Tarmac Quarry Products, Retford). The results given in Table 4-3 to Table 4-5 are the mean of two samples.

Research carried out by BRE [55] explained that the free water is defined as the total water minus the water absorbed by the aggregates and is usually calculated from the aggregates being in the surface saturated surface dry condition (SSD). They reported that it would be impossible to process the aggregate in the mix to obtain SSD condition and thus the technique of pre-wetting the aggregates with some of the mixing water in the mixer pan is advised. This is particularly important when a range of aggregates having different absorption was used. The pre-wetting of the aggregates will allow for most of the effects of the absorption of the aggregates but will not be the same as the absorption of the aggregates measured in accordance to the British Standards. Table 4-2 shows aggregates, which were used in their research, having water absorption that differ according to their sources. They concluded that the calculated water contents and the free water/cement ratio may slightly underestimate the actual free water which was present in the concrete mix. They explained that both the effective (design) water/cement ratio and the total water/cement ratio can be reported in the mix proportions.

Table 4-2: Aggregates water absorption vs. water/cement ratio from BRE [55]

Aggregate-Source	Nominal Size inch	WA %	Effective (Design) W/C	Total W/C
Basalt Shropshire (B1)	3/4 - 3/8	0.6	0.38 - 0.48	0.43 - 0.5
	3/8 - 3/18	0.4		
	3/16 down	0.4		
Granite Carmarthenshire (GT1)	3/4 - 3/8	0.6	0.42 - 0.52	0.45 - 0.55
	3/8 - 3/18	0.7		
	3/16 down	1.1		
Hornfels Westmorland (H1)	3/4 - 3/8	0.1	0.43 - 0.46	0.45 0.48
	3/8 - 3/18	0.2		
	3/16 down	1.1		
Limestone Gloucestershire (L2)	3/4 - 3/8	0.9	0.44 - 0.47	0.47 - 0.51
	3/8 - 3/18	1.0		
	3/16 down	1.1		
Limestone Lancashire (L4)	3/4 - 3/8	0.3	0.45 - 0.58	0.46 - 0.51
	3/8 - 3/18	0.8		
	3/16 down	0.8		
Magnesian Limestone (L9)	3/4 - 3/8	3.1	0.46 - 0.52	0.55 - 0.6
	3/8 - 3/18	4.6		
	3/16 down	1.0		

4.1.3.1 Coarse Recycled Concrete Aggregate (RCCA)

Water absorption and density for RCCA was measured according to the method described on BS 812, Part 2, 1995 [60]. The following equations were used

Equation 4-3: $$\text{Water absorption} = \frac{100 \times (A - D)}{D}$$

Equation 4-4: $$\text{Surface Saturated (SS) Density} = \frac{A}{(A - (B - C))}$$

Equation 4-5: $$\text{Oven dried density} = \frac{D}{(A - (B - C))}$$

Equation 4-6: $$\text{Apparent density} = \frac{D}{(D - (B - C))}$$

Where A is the weight of saturated surface dry aggregate, B is the weight of the glass jar containing the aggregate and topped up with water, C is the weight of the glass jar only topped up with water, and D is the weight of oven dried aggregate. It was expected that the RCCA would have higher water absorption and lower density than the natural limestone because of the attached cement mortar which provides more porosity (i.e. more voids) and thus lesser density to that of natural limestone.

The results for 10-mm size aggregate are shown in Table 4-3 and for 14-mm size aggregate in Table 4-4. The water absorption for 10 mm RCCA is around 75% higher than the water absorption for limestone with similar size, while the SSD is about 7% lower than the SSD of natural limestone with similar size. Similar findings were reported for 14 mm RCCA. The water absorption was found about 75% higher than the corresponding natural limestone aggregate. While for SSD it was found around 4% lower than that for natural limestone aggregate.

Concerning the effects of different crushing methods on water absorption and densities, there were neither distinctive differences nor clear trends, however it is an indicative that there is a significant proportion of mortar remained attached to the recycled aggregate, which lead to higher water absorption.

Similar density values may indicate that there is no significant difference on the amount of cement mortar attached to RCA obtained from the three different crushing machines. In other words, these crushers (if the feed materials are similar) could possibly produce RCCA with approximately similar percentages of attached mortar to the aggregate particles.

Table 4-3: Water absorption and densities for 10 mm RCCA*

Crusher Type	Cone	Impact	Jaw	Limestone
Water absorption	4.6 %	6.0 %	5.3 %	1.3 %
Density SSD kg/m^3	2446	2461	2426	2641
Density O.D kg/m^3	2338	2321	2303	2606

Table 4-4: Water absorption and densities for 14 mm RCCA*

Crusher Type	Cone	Impact	Jaw	Limestone
W.A	4.4 %	4.1 %	4.9 %	1.1 %
Density SSD kg/m^3	2702	2484	2439	2646
Density O.D kg/m^3	2588	2386	2325	2617

Table 4-5: Water absorption and densities for RCFA*

	Cone	Impact	Jaw	Gravel Sand
Water Absorption	6.8 %	5.8 %	6.7 %	1.7 %
Density (SSD) kg/m^3	2387	2385	2448	2627
Density (O.D) kg/m^3	2236	2254	2296	2584

* See Appendix 2 for all the results.

However, this could not be considered as a definite conclusion as other methods [12] could be used to measure accurately the amount of attached mortar for these recycled aggregates. It was not aimed to measure it in this research nor to improve the quality of aggregate.

4.1.3.2 Fine Recycled Concrete Aggregate (RCFA)

The water absorption for RCFA is higher than that for natural river gravel sand. This is also due to the attached paste. The results given in Table 4-4 show that the water absorption for RCFA is around three times greater than that of natural gravel sand of similar maximum size and grading profile. The density results show that the SSD is 9% lower than that of natural gravel sand of similar size.

There is no significant difference between the values of water absorption for RCFA obtained from the different crushing methods. This might also indicate that these crushers (if the feed materials are similar) could produce RCFA with approximately similar percentages of attached mortar to the aggregate particles.

4.2 Strength of recycled concrete aggregate

It is difficult, and often meaningless, to test the compressive strength of individual particles of aggregate. The most common method is to compact aggregates in bulk or use other indirect methods such as the ten percent fines value (TFV) test. Because the RCA in this project was obtained from a parent concrete of known high strength, it was considered unnecessary to measure the compressive strength of the RCA. However, because of the varied shape and uncertain effects of the angularity or flakiness of the RCA, a TFV test was carried out. An aggregate impact value (AIV) test was also carried out for completeness. The TFV test was carried out according to BS 812, Part 111, 1990[61] and AIV test according to BS 812, Part 110, 1990[62]. The tests were carried out on 14-mm coarse RCA size. The results are the mean of two tests.

4.2.1 Aggregate impact value test (AIV)

The Aggregate Impact Value gives a relative measure of the resistance of an aggregate to the sudden shock of impact. The test is of particular value in evaluating an aggregate that is thought to be brittle, but its most extensive use is as an alternative to the aggregate crushing value test.

Resistance to impact of a sample of aggregate in a surface dry condition is measured by subjecting a 28mm deep bed of 14 mm to 10 mm chippings, in a 102mm diameter hardened-steel cap, to 15 blows from a 13.5 to 14.0 kg hammer falling from a height of 380 mm. The percentage by weight of fines passing the 2.36mm BS Test Sieve formed in the test is known as the "Aggregate Impact Value". The results shown in Table 4-6 reveal that the values are almost identical for the RCCA that derived from three different crushing methods, around 24%.

This agrees with Hasaba (19) who reported it to be 23%. However Hansen and Narud(12) reported the AIV for RCCA to be around 20% but the size of RCCA were in this range 16-32 mm. The natural limestone value is 20 %. BS 882:1992(28) limits the maximum values for AIV when the aggregate used for (a) heavy duty floor 25%, (b) wearing surface 30%, and (c) other uses 45%. As with the TFV all aggregates are suitable for all the above conditions.

Table 4-6: Values of Aggregate Impact Test & Ten Percent Fines Test for recycled aggregate derived from different crushers

Crusher Type	Cone	Impact	Jaw	Natural Limestone
A.I.V. %	25	23	24	20
T.F.V. kN	110	170	160	150

4.2.2 Ten percent fines value test (TFV)

Resistance to crushing of a sample of aggregate is measured by submitting a sample to an appropriate load in a compressive testing machine. The load is adjusted to give Ten Percent Fines and is reported in Kilonewtons force.

This procedure ensures that different samples are crushed to the same extent and so overcomes the deficiencies of the aggregate crushing value test, which is insensitive to weak aggregates. The Ten Percent Fines test is equally suitable for the strongest and weakest aggregates and has all but virtually replaced the aggregate crushing value test. It yields results which range from about 400 kN for the strongest aggregates and down to 10 kN or less for weak materials.

Considering the origin of the RCCA, which is identical, this test showed some differences between the crushing machines as shown in Table 4-6. BS 882:1983 limits the minimum values for TFV when the aggregate used for (a) heavy duty floor 150 kN, (b) wearing surface 100 kN, (c) other uses 50 kN. This would qualify the RCCA obtained from impact and jaw crusher to be used in any type of concrete, and exclude the cone crushed RCCA from heavy-duty floors. Lower values for cone crusher is in consistent with higher values of flakiness index RCA from cone crusher as reported earlier. This is comparable to Hasaba *et al*(9) who reported TFV values within a similar range 11.3 tons (111 kN) to 13.3 tons (130 kN).

4.3 Grading of recycled concrete aggregate

After crushing the slab units, the recycled aggregate were separated into three portions; 14 mm single size, 10 mm single size aggregate and fine recycled aggregate (passing sieve size 5 mm). They were chosen because precast concrete hollow core slab units are produced commercially using these portions. The grading of these aggregates were compared with the BS 882, 1992(28) as well as with natural limestone coarse aggregate and natural sand.

4.3.1 Coarse recycled concrete aggregate

Two portions of the recycled coarse concrete aggregate (RCCA) were obtained 14 mm and 10 mm. Figure 4-2 shows the grading curve for 14 mm single size aggregate obtained from three crushers. All grading complied with BS 882, 1992 [28] for single sized aggregate, although the grading for the jaw and cone crushed RCA lie closer to the BS limits than the impact crushed RCA and the natural aggregate. However, it would possible to adjust the crushing and sieving operation to produce better grading. For 10 mm single size, Figure 4-3 shows that all RCCA obtained from the three crushers have similar grading and also complied with BS 882, 1992 [28].

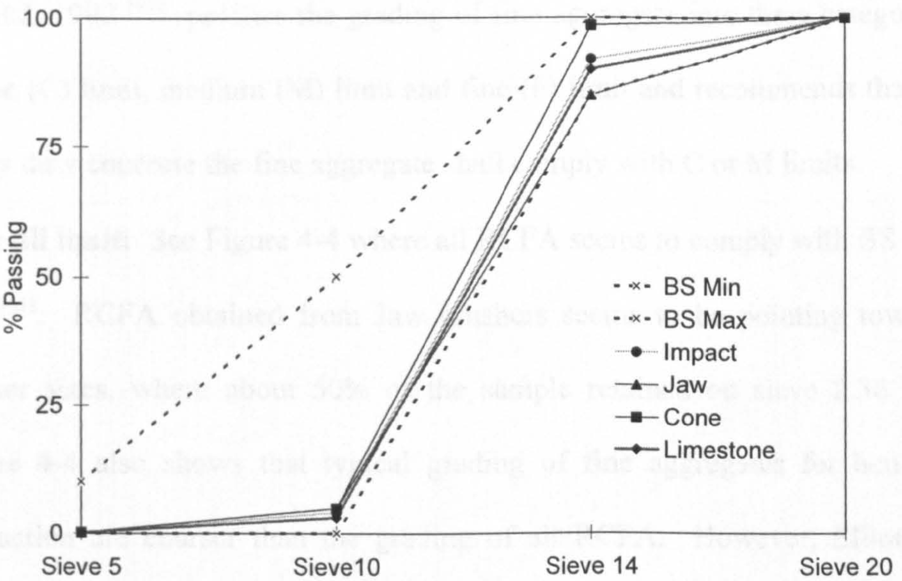


Figure 4-2: RCCA 14 mm single size grading curve derived from different crushers

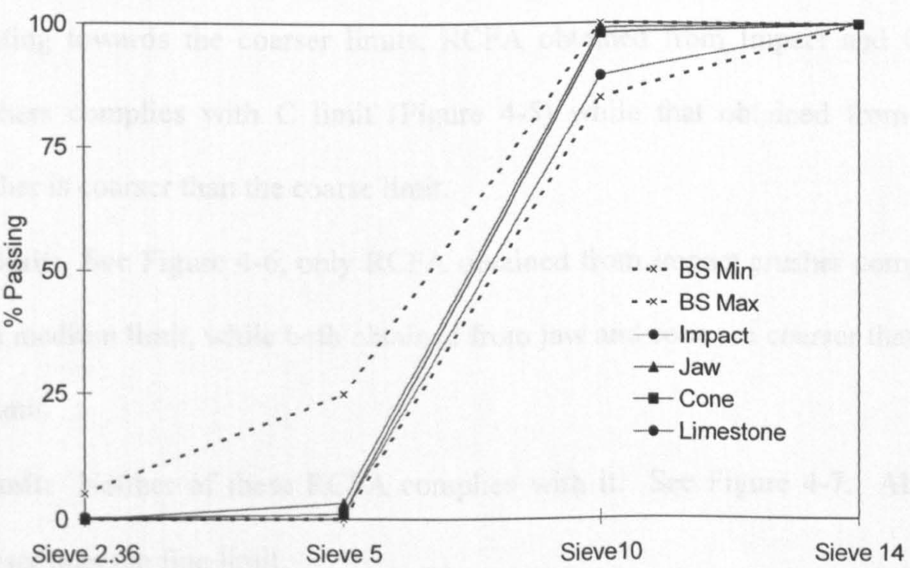


Figure 4-3: RCCA 10 mm single size grading curve derived from different crushers

4.3.2 Fine recycled concrete aggregate

BS 882, 1992 [41] specifies the grading of fine aggregate into three categories: coarse (C) limit, medium (M) limit and fine (F) limit and recommends that for heavy duty concrete the fine aggregate shall comply with C or M limits.

Over all limit: See Figure 4-4 where all RCFA seems to comply with BS 882, 1992(41). RCFA obtained from Jaw crushers seems to be pointing towards coarser sizes, where about 50% of the sample retained on sieve 2.36 mm. Figure 4-4 also shows that typical grading of fine aggregates for hcu UK production are coarser than the grading of all RCFA. However, Elliott [94] found that fine aggregates with high portions passing sieve 600µm are linked to hcu with poor bonding, see Figure 4-4, more about bonding are in chapter 7.

C Limit: All recycled fine aggregate obtained from the three crushers are pointing towards the coarser limits. RCFA obtained from Impact and Cone crushers complies with C limit (Figure 4-5) while that obtained from jaw crusher is coarser than the coarse limit.

M limit: See Figure 4-6, only RCFA obtained from impact crusher complies with medium limit, while both obtained from jaw and cone are coarser than the M limit.

F limit: Neither of these RCFA complies with it. See Figure 4-7. All are coarser than the fine limit.

In summary the RCFA has a rather coarse grading with only about 20% passing the 600µm sieve, as opposed to the more usual figure of 30% to 35% in quarried sands. This has a significant effect of mix design as the proportion of fine aggregate required to maintain constant workability would need to be considerably increased, typically by about 20%. The proportion of fines passing the 300μm sieve is more than desirable, although the dust was removed from the RCFA during the screening process during crushing.

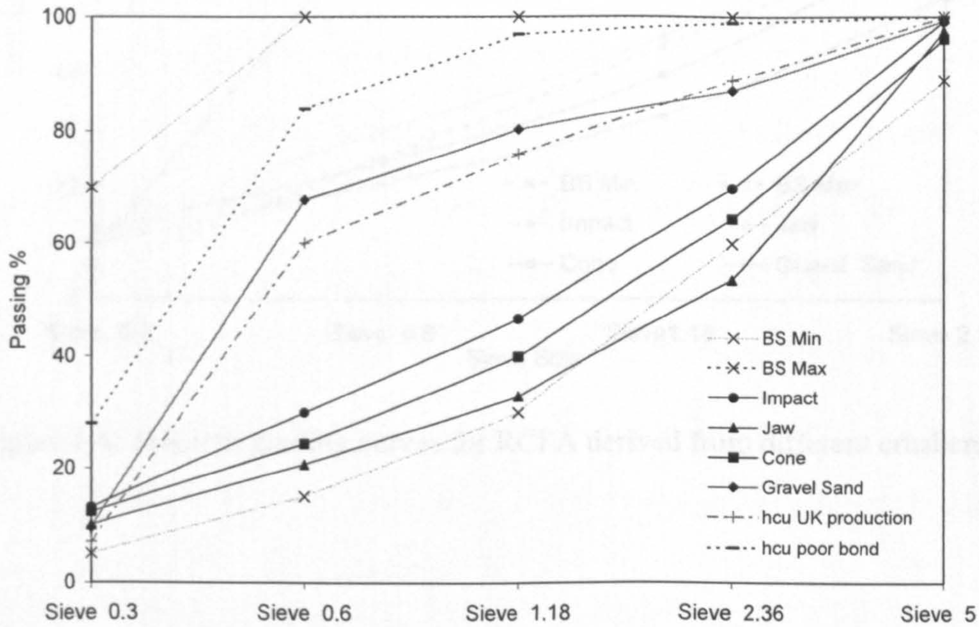


Figure 4-4: Overall grading curves for RCFA derived from different crushers

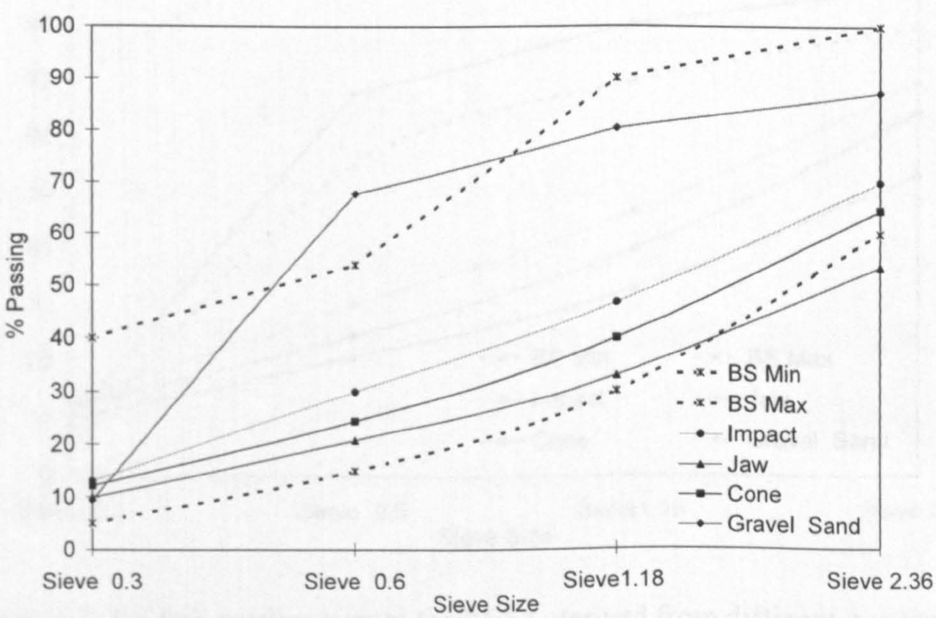


Figure 4-5: C-Limit grading curves for RCFA derived from different crushers

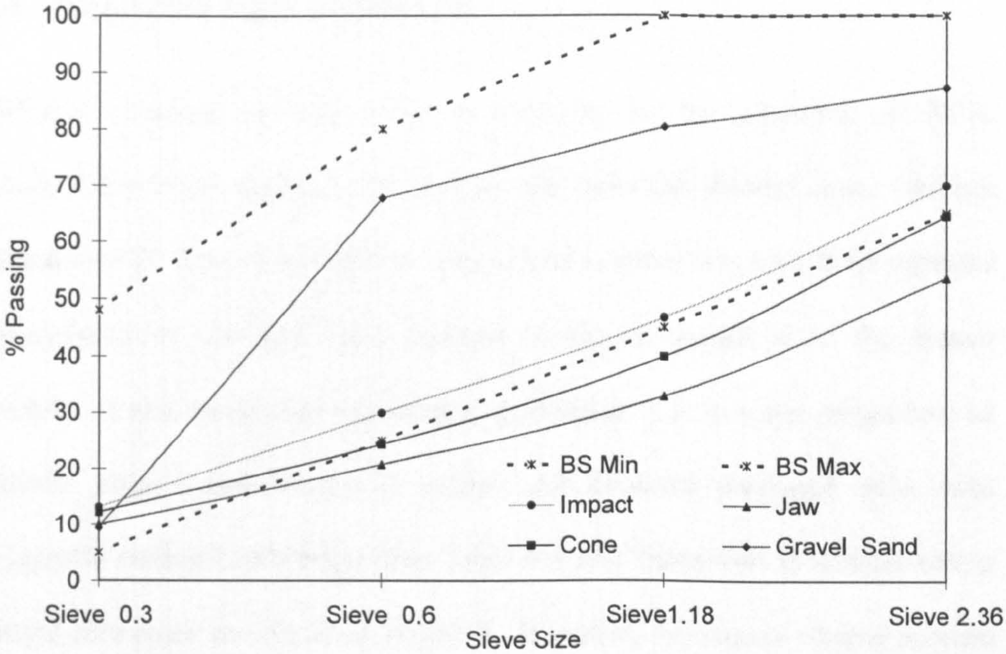


Figure 4-6: M-Limit grading curves for RCFA derived from different crushers

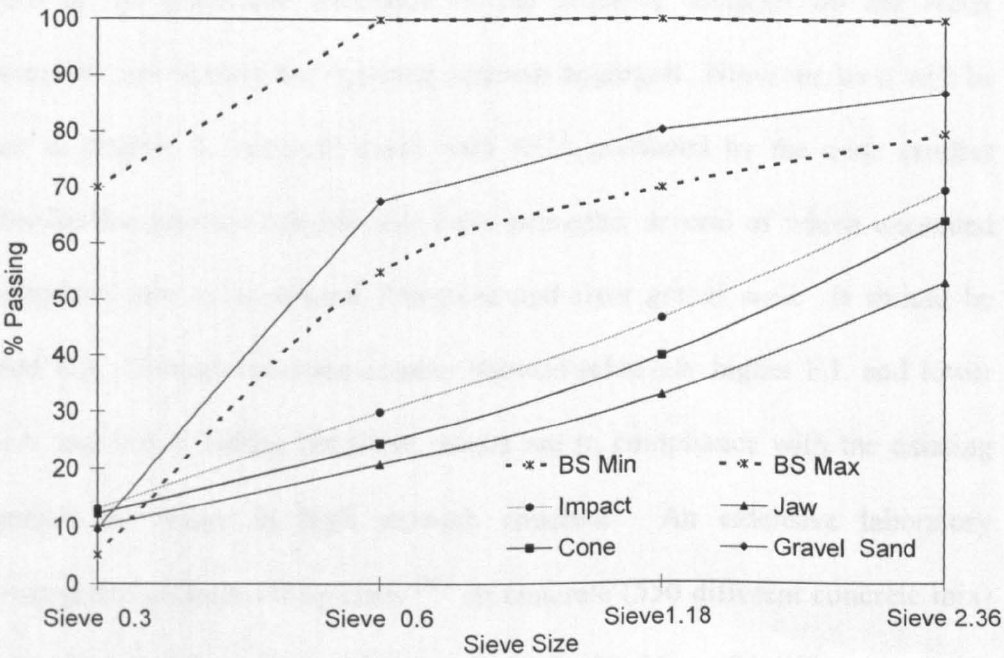


Figure 4-7: F-Limit grading curves for RCFA derived from different crushers

4.4 Discussion and Conclusion

Different crushing methods have an influence on the properties of RCA. Physical properties appear to be affected the most and showed some variance related to each type of crusher, in which their mechanisms vary from repeated pressures as in jaw and cone crushers to kinetic impact as in the impact crusher. It was found that one crusher performed well in some properties but showed some disadvantages in others. All crushers produced RCA with acceptable strength and shape (See Table 4-1 and Table 4-6) in comparison to natural limestone and common standard. However, the impact crusher appears to be the most suitable overall by producing RCA with better shape (lower F.I. values) and strength (better A.I.V and T.F.V values). This is followed by the jaw crusher and then the cone crusher.

There is no distinctive influence of the crushing methods on the water absorption and density for recycled concrete aggregate. However, as it will be seen in chapter 5, concrete made with RCA produced by the cone crusher achieved the greatest compressive cube strengths, several of which exceeded the control mix using natural limestone and river gravel sand. It should be noted that although the cone crusher showed relatively higher F.I. and lower A.I.V and T.F.V values but these results are in compliance with the existing standard for usage in high strength concrete. An extensive laboratory investigation carried out by BRE [55] on concrete (550 different concrete mix) made with natural crushed rock aggregates obtained from 24 different quarries all over the UK came to a conclusion that the difference in the performance of concrete could not be related to any single characteristics of the aggregates. Nevertheless BRE [55] also concluded that good quality concrete could be made with all the aggregates used.

Comparing RCA hcu aggregates with the natural crushed aggregates used in BRE [55] research, the RCCA was found to have similar (or even better) properties. Flakiness values for RCCA (from all crushers) are lower than the average value (around 24) for all crushed rock obtained from the 24 quarries, while the angularity number for crushed rock used in BRE research, which varied from 7 % for Durham Dolomitic Limestone to 13% for Cornwall Granite, also showed similar values to RCCA (given in Table 4-1) which varies from 6% to 11%.

However, RCCA found to differ considerably in water absorption, reported to absorb about 4 to 6 times more water than the natural crushed limestone mainly due to the more porous attached mortar. These findings are consistent with Hansen and Narud [12] Mulheron and O'Mahony[20] Hasaba *et al* [19]. The high porosity of RCA can lead to a reduction in the concrete density, but the reduction in density was found to have minimal effect on the compressive and flexural strength of concrete; these effects will be discussed further in later chapters. The high water absorption could also affect the workability of fresh concrete but combining it with other different aggregate characteristics i.e. shape (flakiness and angularity) and texture of RCA, may increase the complexity of defining and measuring the effects on workability, as will be seen in next chapter.

All RCA appear to have a reasonable grading comparing with the existing British Standard and is similar to that for natural limestone aggregate. However, it should be noted that the impact crusher has one major disadvantage, which is that it has a large reduction factor (from the feed to the output) and consequently it produces large amount of fine aggregate than coarse aggregate; this agrees with Boesman's findings [54], however the crusher setting could be adjusted to minimise such disadvantages.

The fine RCFA was considerably coarser than the natural river gravel, and technically did not comply with the BS coarse category limits, failing at the 2.36mm sieve size only. The effect of a smaller fraction of RCFA below 600 μm may have a significant effect on the desired mix proportions to keep workability constant. In spite of the generally poor characteristics of the RCFA, in terms of grading, the effect on the workability and strength of the resulting concrete was not greatly deleterious, even at 50% replacement, as it will be discussed in next chapters.

4.4.1 Conclusion

In conclusion all three crushers could be used to produce acceptable shape and strength of coarse RCA and in most properties are competitive to that of natural crushed aggregates which were obtained from 24 different quarries all over the UK [55]. It should be noted that one crusher performed well in some properties and shows some disadvantages in others but this could easily be improved by adjusting the setting of the crushers accordingly.

Apart from the fact that impact crusher produces a sufficient amount of RCFA, it showed relatively better performance then followed by jaw and then cone. Concerning the shape and strength, RCA showed similar properties, and in some cases better than the conventional limestone aggregate apart from the fact that RCA absorbs water from 4 to 6 times than of the natural aggregate. RCA derived from the three crushers will continue to be used in the following chapter to study their effects on fresh concrete properties.

5 Mix Design and Workability of Concrete with RCA

Introduction

This chapter outlines the mix design that was used for both conventional concrete and concrete with recycled aggregate. For all mixes the mix proportions and mix procedures were kept constant and in compliance with the Department of Environment, Design of Concrete Mixes [63], ensuring that a fair comparison would be achieved between conventional concrete and concrete with recycled aggregate obtained from different crushers using rejected hollow core slab units (hcu).

Properties of fresh concrete, especially the workability, were studied and measured using two most common methods - the slump and the compacting factor. The strength of hardened concrete; i.e. compressive, flexure and tensile splitting strength, were investigated in the following chapter.

5.1 Mix Design

A high strength mix design was chosen because the parent of the RCA was from hcu and that is of high grade and uncontaminated material with compressive cube strength between 50 and 80 N/mm^2 at 28 days. The intention is also that these high quality RCA could be reused within the precast concrete industry leading to a greater efficiency and substantial reduction in the demand for natural aggregates and consequently easing the impact on landfills and the environment.

The natural control mixes were chosen as a reference to compare with because they are well established and widely used in the concrete industry and in compliance with the British Standard, while recycled aggregates standards and full recognition are yet to be established.

The proportion for the control mix with 100% natural aggregates is given in Table 5-2. The natural coarse limestone aggregate is crushed carboniferous limestone obtained from Retford and the fine aggregate is natural sand. The control mix was designed to reach about 60 N/mm^2 target compressive strength, similar to the parent of the recycled aggregates. It should be noted that it is not possible to use the same mix design as the parent one (see Table 3-2) because that is a semi-dry mix intended for an extrusion machine used to manufacture hcu.

The design water/cement ratio was kept constant for all mixes and for a clearer comparison no additions or admixtures were used. The only variable is the replacement percentages of RCA obtained from different crushers.

The Department of Environment, Design of Concrete Mixes [63] recommended that the aggregates to be batched in an oven-dried condition and extra water should be added to compensate for the water absorption of aggregates to enable it to reach the saturated surface dry (SSD) condition, Table 5-1 is the amount of extra water to be added (for 1000 kg batch) as recommended by the Department of Environment, Design of Concrete Mixes [63]. However in reality the SSD condition may not happened, as the extra water may not be fully absorbed during the mix by the aggregates; whether for the natural aggregates mix or the RCA aggregates mix.

There is a likelihood that some amount of the added water may remain free to have some bearing on the W/C ratio. It should be noted that this phenomenon affects <u>both</u> natural crushed aggregate and RCA. Earlier research work was carried out by BRE [55] to prove that the natural crushed rock aggregates can play a major role to overcome the shortage of gravel aggregates supply in England. Based on 550 concrete mixes made with 24 different types of natural crushed rock aggregates, they reported that it would be impossible to process the aggregates during the mix casting to achieve a surface saturated condition and acknowledged that the noted water/cement ratio in their work may slightly underestimate the actual water that was present in the concrete mix. They came to the conclusion that a technique of pre-wetting the aggregates with some of the mixing water in the mixer pan is advised to enable the dry aggregates to absorb water; this technique will be discussed further in the next section. It should be noted that their research work was aimed to understand the performance of natural crushed aggregates in concrete and to change limits specified in various British Standards accordingly, and was also used as a reference for the Department of Environment's- Design of Concrete Mixes.

For clarity; and as declared earlier, the scope of this present study is to investigate the RCA aggregates and its performance in comparison to natural aggregates under same exposures according to existing established standards and procedures. Thus, the above BRE research work validates the approach to use the designed water/cement ratio in Table 5-2 to Table 5-4 as a fixed reference to the mix proportions through out this study.

The replacement proportions of RCA were chosen as 20% and 50%. The former represents a typical limit for RCCA proposed in P.I.T project [5]. The latter was chosen because this is thought to be an extremity worthy of consideration in studying the sensitivity of concrete made with such high replacements, especially for the fine aggregate. The replacement was made for (i) coarse aggregate alone, (ii) fine aggregate alone and (iii) coarse and fine mixed. For each crusher three sets of mixes were used. The sets are (i) only RCCA was replaced and it contains two mixes, one for 20% RCCA replacement and the second for 50% RCCA replacement. Set (ii) is for RCFA alone with similar replacement percentages. Set (iii) is for a combination of both RCCA and RCFA with similar replacement percentages. All crushers have similar sets of mixes. Table 5-2, 5-3 and 5-4 show the mixes for the jaw, cone and impact crushers, respectively.

5.1.1 Mix procedure

The aggregates were batched in an oven-dried condition. According to the Department of Environment, Design of Concrete Mixes [63], the weights of the oven dried batched aggregates were obtained by multiplying the aggregate saturated surface dry (SSD) weights by 100/(100+A), where A is the percentage by weight of water needed to bring the dry aggregates to a saturated surface dry condition.

The amount of mixing water should be increased by the weight of water absorbed by the aggregate to reach the saturated surface dry condition [63], and it is called the extra water.

Table 5-1: The amount of extra mixing water added for oven dry aggregate per 1000 kg batch to reach the saturated surface dry condition according to mix standard [63]

Crushers	Control	RCCA		RCFA		RCCA+RCFA	
	M0	M1	M2	M3	M4	M5	M6
	0%	20%	50%	20%	50%	20%	50%
Jaw	10.4	13.9	19.3	13.3	17.7	16.8	26.6
Cone	10.4	13.4	17.9	13.3	17.8	16.4	25.4
Impact	10.4	13.7	18.8	12.8	16.3	16.1	24.8

The values were shown in Table 5-1 (for 1000 kg batch) and were calculated for the mix proportions shown in Table 5-2, 5-3 and 5-4. It should be noted that the actual total weight of each mix in this research was chosen to be equal to the maximum weight capacity of the laboratory mixer and that is 245 kg.

The water absorption of aggregates is obtained from Table 4-3, 4-3 and Table 4-5. For example, in Table 5-1, 10.4 kg extra water would need to be added for a batch of 1000 kg control mix to enable the aggregate to reach SSD conditions. It was calculated as follows:

Extra water for any size (kg) = Weight of aggregates x Water absorption

Extra water for 14 mm = 275 x 1.1% = 3.0 kg

Extra water for 10 mm = 183 x 1.3% = 2.4 kg

Extra water for –5 mm = 292 x 1.7% = 5.0 kg

Total extra water = 3.0 + 2.4 + 5.0 = 10.4 kg

It should be noted that the weights of the aggregates in the tables below are in a saturated surface dry condition. In the actual mixes the weights of oven dried batched aggregates were used and obtained as follows (for 1000 kg batch):

Oven Dried Aggregates = SSD Aggregates x [100 / (100 + W.A)]

14 mm oven dried = 275 (kg) x [100 / (100+1.1)] = 272 kg.

Similar calculations were done for all the mixes as shown in the tables.

British Standard BS 1881, Part 125, 1986 [64] was followed for the mixing procedure, which recommends that the oven dried aggregate is weighed and mixed dry for about 30 seconds. About one-third of the mixing water was added and the mixing continued for 2 minutes. Then the aggregates were covered to stand still for almost 10 minutes to give oven dried aggregate opportunity to absorb water, this process is an attempt to encourage the aggregates to reach SSD condition. After that the cement was added and mixed for 30 seconds and then the remaining water was added and the mixing continued for another 2 minutes.

It was anticipated that the extra water, see Table 5-1, may not be fully absorbed by the oven dried aggregates (to be in SSD) and may remain free to disrupt the designed water to cement ratio. This was verified by BRE [55] research work as discussed earlier in section 5.1. It should be noted that the aggregates were not fully immersed in water during the mixing and some tests were carried out in order to estimate the amount of water that is absorbed by the oven dry aggregate. These tests are aimed to simulate the mixing procedure by adding the mixing water to the aggregates for 10 minutes, then the aggregates were drained and oven dried to estimate the amount of absorbed water, see Appendix 2 for tests details. It is found that, within 10 minutes, natural aggregates absorbed an average of 70% of the total mixing water while the RCA absorbed around 90% of the total mixing water.

Table 5-2: Mix proportions by weight of 1000 kg (& by volume kg/m^3) for control concrete & concrete with recycled aggregate obtained from jaw crusher.

Material	Mix Proportions						
	Control	RCCA		RCFA		RCCA+RCFA	
	M0	M1	M2	M3	M4	M5	M6
	0%	20%	50%	20%	50%	20%	50%
O.P. Cement	167 (404)	167 (399)	167 (400)	167 (401)	167 (398)	167 (401)	167 (395)
Water	83 (201)	83 (198)	83 (199)	83 (199)	83 (198)	83 (199)	83 (196)
14 mm NA*	275 (665)	220 (526)	138 (329)	275 (660)	275 (656)	220 (528)	138 (325)
10 mm NA*	183 (442)	147 (350)	92 (219)	183 (439)	183 (436)	147 (351)	92 (216)
Sand*	292 (706)	292 (698)	292 (699)	233 (561)	146 (348)	233 (561)	146 (345)
14 mm RCCA*	-	55 (131)	138 (329)	-	-	55 (132)	138 (325)
10 mm RCCA*	-	37 (87)	92 (219)	-	-	37 (88)	92 (216)
RCFA*	-	-	-	58 (140)	146 (348)	58 (140)	146 (345)

Table 5-3: Mix proportions by weight of 1000 kg (& by volume kg/m^3) for control concrete & concrete with recycled aggregate obtained from cone crusher.

Material	Mix Proportions						
	Control	RCCA		RCFA		RCCA+RCFA	
	M0	M1	M2	M3	M4	M5	M6
	0%	20%	50%	20%	50%	20%	50%
O.P. Cement	167 (404)	167 (403)	167 (399)	167 (405)	167 (402)	167 (403)	167 (394)
Water	83 (201)	83 (200)	83 (198)	83 (201)	83 (200)	83 (200)	83 (196)
14 mm NA*	275 (665)	220 (531)	138 (328)	275 (666)	275 (661)	220 (531)	138 (325)
10 mm NA*	183 (442)	147 (535)	92 (219)	183 (443)	183 (440)	147 (353)	92 (216)
Sand*	292 (706)	292 (704)	292 (697)	233 (566)	146 (351)	233 (563)	146 (345)
14 mm RCCA*	-	55 (133)	138 (328)	-	-	55 (133)	138 (325)
10 mm RCCA*	-	37 (88)	92 (216)	-	-	37 (88)	92 (216)
RCFA*	-	-	-	58 (142)	146 (351)	58 (141)	146 (345)

Table 5-4 Mix proportions by weight of 1000 kg (& by volume kg/m^3) for control concrete & concrete with recycled aggregate obtained from impact crusher.

Material	Mix Proportions						
	Control	RCCA		RCFA		RCCA+RCFA	
	M0	M1	M2	M3	M4	M5	M6
	0%	20%	50%	20%	50%	20%	50%
O.P. Cement	167 (404)	167 (402)	167 (398)	167 (400)	167 (399)	167 (400)	167 (396)
Water	83 (201)	83 (200)	83 (198)	83 (199)	83 (199)	83 (199)	83 (197)
14 mm NA*	275 (665)	220 (529)	138 (328)	275 (659)	275 (658)	220 (527)	138 (326)
10 mm NA*	183 (442)	147 (352)	92 (218)	183 (438)	183 (438)	147 (351)	92 (217)
Sand*	292 (706)	292 (702)	292 (696)	233 (559)	146 (349)	233 (559)	146 (346)
14 mm RCCA*	-	55 (132)	138 (328)	-	-	55 (132)	138 (326)
10 mm RCCA*	-	37 (88)	92 (218)	-	-	37 (88)	92 (217)
RCFA*	-	-	-	58 (140)	146 (349)	58 (140)	146 (346)

* All Aggregates in a saturated surface dry condition.

Other researcher have attempted to study this further; tests were done by Evangelista and Brito[65] who reported that there was a reduction in the compressive strength in mixes where its aggregates were soaked for longer periods. They related this to the longer pre-wetting procedures leading to a higher water absorption by the aggregates and thus the mechanical connections between the aggregates and the cement paste were shallower and mechanical strength lower. Furthermore Hansen [6] explained that the time needed to saturate recycled aggregate by soaking fully in water was 15 minutes for coarser recycled aggregate (4-28 mm), 10 minutes for 2-4 mm size, and 5 minutes for finer 0-2 mm size. He also reported that it is necessary to pre-soak the RCA in order to prevent a rapid decrease in workability. He explained that soaking the aggregate in water for one hour before mixing was sufficient, but added that there was little difference in compressive strength of concrete made with air dry RCA and that with saturated surface dry RCA when the W/C of fresh concrete was the same.

On the other hand Hansen and Marga [22] reported difficulties in establishing the stage at which the mixed aggregates are in surface saturated conditions because both fine and coarse aggregates were mixed. It was experienced that coarser size aggregates tend to be extra dry (exceed the SSD stage) while the fine aggregates were still wet by trapping the water between its particles. Neville [59] also reported that the demarcation between absorbed water and the free water is difficult because the absorption of water by dry aggregates slows down or is stopped owing to the coating of particles with cement paste during the mix and suggested that it is would be useful to determine and use the quantity of water absorbed in 10 to 30 minutes instead of the total water absorption (to SSD) which may never be achieved.

5.2 Fresh Concrete Properties

5.2.1 Workability

Workability refers to the properties of the fresh concrete before it sets and hardened. Although the subject is complex and difficult to define and measure, a few researchers [55] have shown that aggregate's characteristics such as grading, shape and surface texture all have an effect on workability. The methods used in this study were slump according to BS 1881, Part 102, 1991[66] and compacting factor (CF) according to BS 1881, Part 103, 1991[67]. Although the slump method is simple, it is widely used and easily understood measurement of workability and therefore used here. The CF method was also used because it gives a better understanding of mixes of relatively low and medium workability. VB test was not a preferable choice as it is only applicable to low workability and would not be suitable for some mixes that shows high workability in this research. Table 5-5 shows that some tests are sensitive for lower workability, whilst others are more suitable for higher workability. Since none of the tests is particularly sensitive for both very low and very high workability, another different test Tattersall's Two-point test [68], was attempted. Unfortunately, obtaining readings from the machine for all the mixes was unsuccessful. Some mixes in this research were too stiff for Tattersall's mixer machine and for this reason it was decided not to continue using this workability method and the efforts were refocused on the slump and compacting factor.

Table 5-5: Tests methods appropriate to mixes of different workability according to BS 1881:1983

Workability	Methods
Very low	Vebe time
Low	Vebe time, Compacting factor
Medium	Compacting factor, Slump
High	Compacting factor, Slump, Flow table
Very high	Flow table

5.2.1.1 Slump Methods

The results are given in Figure 5-1 and Table 5-6, the target slump for the control mix was 60 mm; see Appendix 2 for details and tests repeatability. It was designed to reach a compressive strength of 60 N/mm^2. Although the slump method is simple, it shows that the general trend is an increase in slump as the RCA replacement increases up to 20%, and with the exception of RCCA replacement where the slump increases at replacement levels greater than 20%, which is followed by a decrease at 50% replacement.

This effect could be related to the surface area-to-volume ratio, which is lower for coarse aggregate than fine and in consequence coarse aggregates absorb less amount of the extra water and this increases the slump.

For the mixes with RCFA alone and for the RCCA and RCFA combined, the slump increased from 60 mm up to 100–125 mm at 20% replacement. Similar trends are shown in Figure 5-2, that presents the effects of extra water against the slump measurements. It confirms what was reported earlier that the slump for concrete with RCFA initially increased at 20% replacement and then tends to decrease when replacements are increased, similarly for the RCCA and RCFA combined. However, it differs for concrete with RCCA where the slump increased when the extra water increased. Table 4-3 to 4-4 show water absorption for RCFA is around an average of 6.4% and this amount is approximately 30% more of the amount of water absorbed by RCCA, an average of 4.8% (combining size 10mm and 14mm). Higher replacement of RCFA causes the increase of water intake and thus causes a less workable mix. This was in line with work carried out by BRE [55] on fine crushed rock obtained from different quarries. They have studies the effects of fine aggregates on the workability of concrete and found that when reducing the fine aggregates contents by 10% there was a considerable reduction in the required water content by around 10kg/m^3 to achieve the required workability. They relate that to higher water intake by the fine aggregates.

Other reports in the literature also tend to adjust the water requirements for the RCA mix in order to achieve the same slump as that for control mix. No mention was made in determining the amount of extra water that was absorbed by the aggregates in their mixes. Ravindrarajah [31], Rasheeduzzafar [32] and Hansen [12] all find similar results and draw the same conclusions, where if both RCCA and RCFA were used then an estimated 25 l/m^3 of extra water was required to achieve similar slump to that of control mix. Mukai [30] reported that if RCCA were used with natural sand then 10 l/m^3 of extra water should be added to achieve a similar slump to that of the control mix. Hansen and Marga [22] increased the mixing water by 23 l/m^3 to achieve the same slump as control mix, but they differ in others in that they increased the cement content accordingly to maintain the same effect of W/C as standard concrete. It should be noted that in this research the extra water was calculated and added based on the water absorption of the RCA according to the mix design manual for concrete [63] rather than attempting to match the workability to that of natural aggregates, for details see Section 5.1.1.

The effect on workability of the different crushing methods on concrete made with the coarse and fine RCA in terms of slump is confusing and contradictory. Changes in the slump value for RCFA are greatest of all, especially for the cone and jaw crushing methods. RCFA crushed in this manner have greater water absorption see Table 4-5. So one would have expected a reduction in slump, which is not seen until the replacement is 50%. The effect of grading, in particular the low fraction passing the 600 μm sieve, would suggest a reduction in workability as the replacement RCFA increases for a fixed ratio of fine-to-coarse aggregate.

For the RCCA, it is the impact crushing method that sees the greatest change. This confirms the results in Table 4-1 where the RCCA obtained from the cone crusher gave the greatest angularity and therefore its effect on workability would be greater as more bleed water might be retained. However, it contradicts the water absorption result Table 4-3 where the impact crushed RCCA had the greatest absorption value, suggesting a reduction in slump. These findings are in agreement with BRE [55] research work who concluded that the workability for concrete mix depends on complex combination of grading; particle shape and surface texture of the aggregates further will be discussed in section 5.3.

Table 5-6: Average Slump values (mm) for concrete with recycled concrete aggregate derived from different crushers (see Appendix 2 for full data)

Replacement Percentages	Concrete with RCCA Derived from			Concrete with RCFA Derived from			Concrete with both RCCA & RCFA Derived from		
	Cone	Impact	Jaw	Cone	Impact	Jaw	Cone	Impact	Jaw
0%	60	60	60	60	60	60	60	60	60
20%	70	75	80	120	120	125	90	100	75
50%	75	125	75	45	85	55	30	30	30

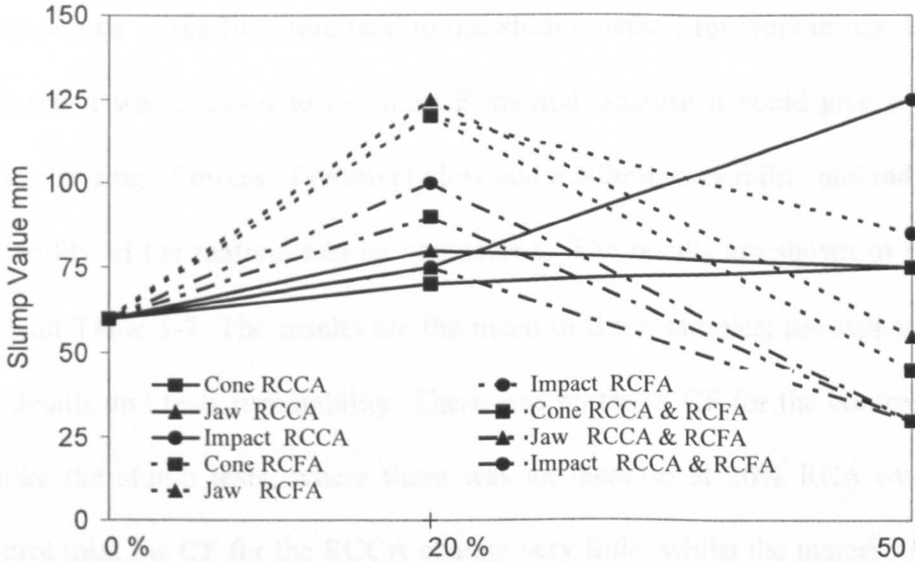


Figure 5-1: Slump values for concrete with recycled aggregate (with different replacement percentages) obtained from different crushers

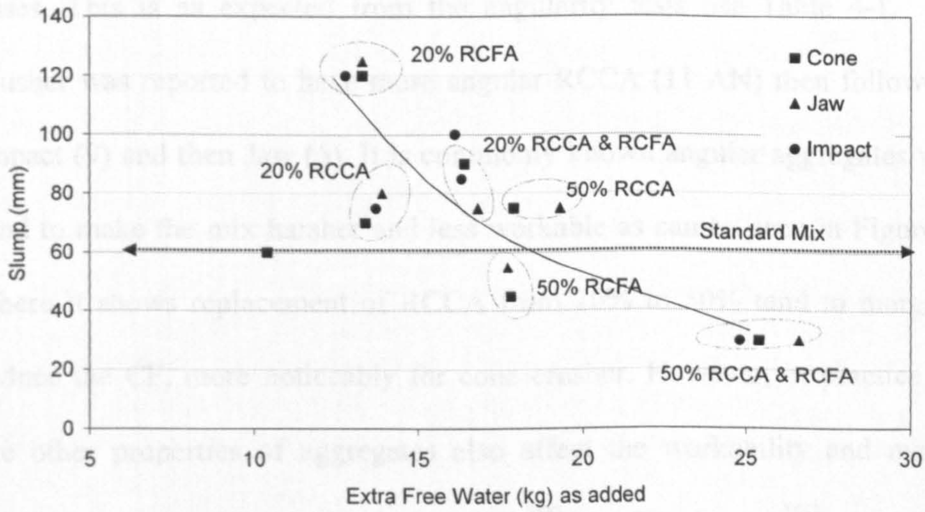


Figure 5-2: Effects of extra water on slump workability

5.2.1.2 Compacting Factor Method (CF)

Most reports in the literature tend to use slump method for workability. In this research it was decided to use the CF method because it could give a better understanding of mixes of relatively low and medium workability and indicates the ability of the materials to be compacted. The results are shown in Figure 5-3 and Table 5-7. The results are the mean of three samples; see Appendix 2 for details and tests repeatability. There was no target CF for the control mix. Unlike the slump tests, where there was an increase at 20% RCA over the control mix, the CF for the RCCA change very little, whilst the majority of the RCFA and RCFA+RCCA reduce considerably beyond the 20%. The exception is that there is very little change in the jaw and impact crushed RCCA.

The influence of the crushing method appears to be more consistent than in the slump results. The CF for cone crushed RCCA is considerably lower in all cases. This is as expected from the angularity tests see Table 4-1. Cone crusher was reported to have more angular RCCA (11 AN) then followed by Impact (9) and then Jaw (6). It is commonly known angular aggregates would tend to make the mix harsher and less workable as can be seen in Figure 5-5; where it shows replacement of RCCA from 20% to 50% tend to marginally reduce the CF, more noticeably for cone crusher. However, in practice there are other properties of aggregates also affect the workability and may not single out certain properties as Tattersall [68] and Teychenne [55] concluded in their extensive research.

When adding RCFA and RCCA together the CF was reduced at a higher rate than in the RCCA alone, see Figure 5-6. The effect of RCFA replacement alone on the CF is as expected, see Table 5-7, i.e. a reduction in CF from 0.98 to 0.93 (average), owing to increased water absorption together with coarser shape of RCFA. This was demonstrated clearly in Figure 5-4 showed that the extra water did not clearly improve the workability in terms of compacting factor. This is could be linked to what has been reported earlier, that the water intake of RCFA is higher (30% approximately) to that of coarser aggregates. At lower amount of extra water (i.e. lower replacements of RCA) the CF remained around the control mix value, but when the amount of extra water increased (higher replacements of RCA) the CF decreased. This was also in line with work carried out by BRE [55] on fine crushed rock obtained from different quarries where they found that when reducing the fine aggregates contents by 10% there was a considerable reduction in the required water content by around 10kg/m^3 to achieve the required workability. Figure 5-9 shows that the Slump and CF patterns are consistent, i.e. when slump decreases the compact factor also decreases, which agrees with Neville[59] and thus it shows of no reasons of not using slump and compacting factor to measure workability for concrete with RCA.

Table 5-7: Compacting factor values for concrete with recycled concrete aggregate derived from different crushers (see Appendix 2 for full data)

Replacement Percentages	Concrete with RCCA Derived from			Concrete with RCFA Derived from			Concrete with both RCCA & RCFA Derived from		
	Cone	Impact	Jaw	Cone	Impact	Jaw	Cone	Impact	Jaw
0%	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98
20%	0.96	0.97	0.98	0.97	0.99	0.98	0.93	0.98	0.96
50%	0.95	0.97	0.97	0.92	0.97	0.93	0.86	0.93	0.93

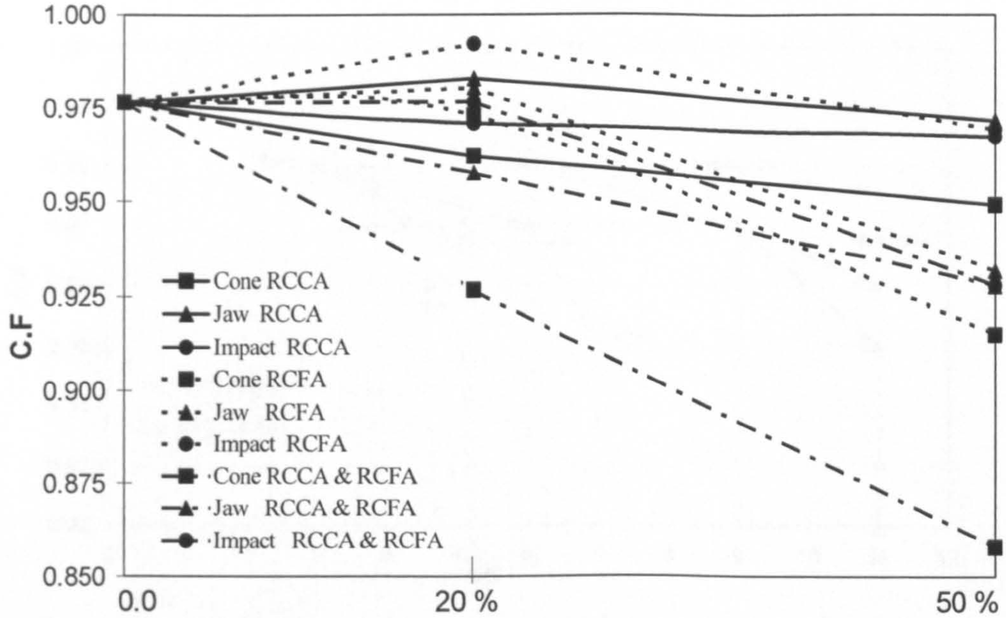


Figure 5-3: Compacting factor values for concrete with recycled aggregate (with different replacement percentages) obtained from different crushers

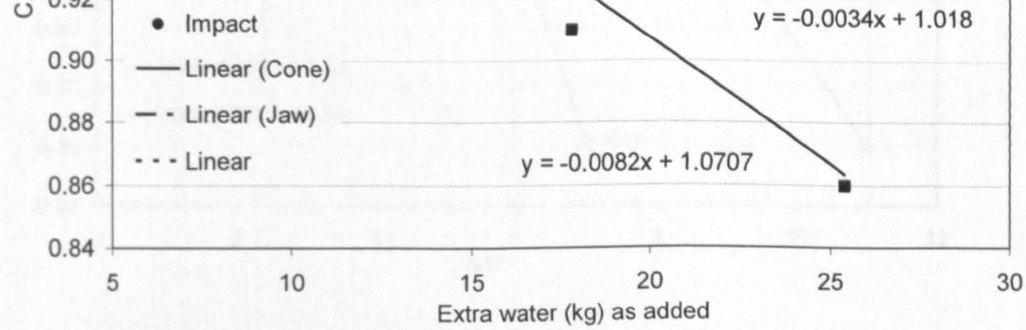


Figure 5-4: Effects of extra water on C.F. workability

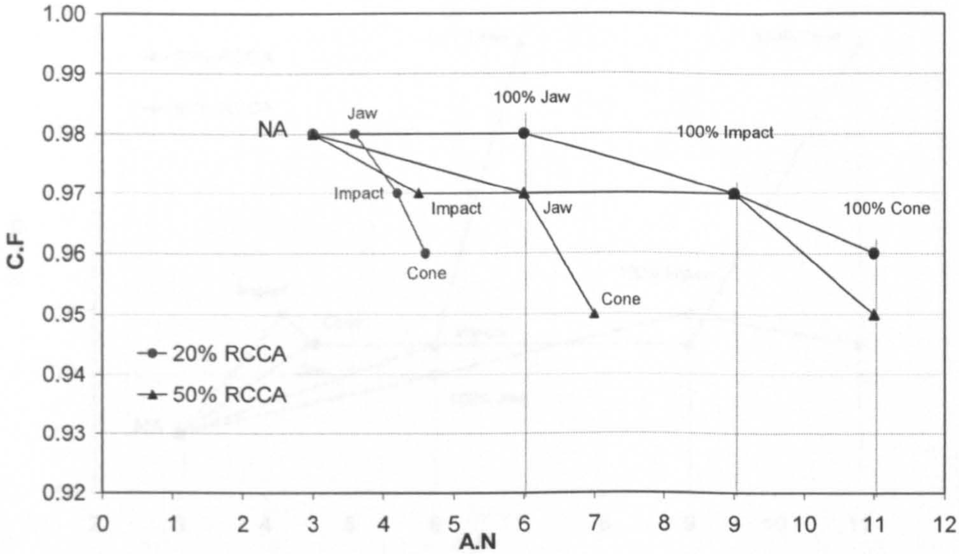


Figure 5-5: Effects of aggregates angularity number on CF for concrete with proportions of RCCA (A.N values for 20% and 50% are derived linearly from 100% laboratory tested value)

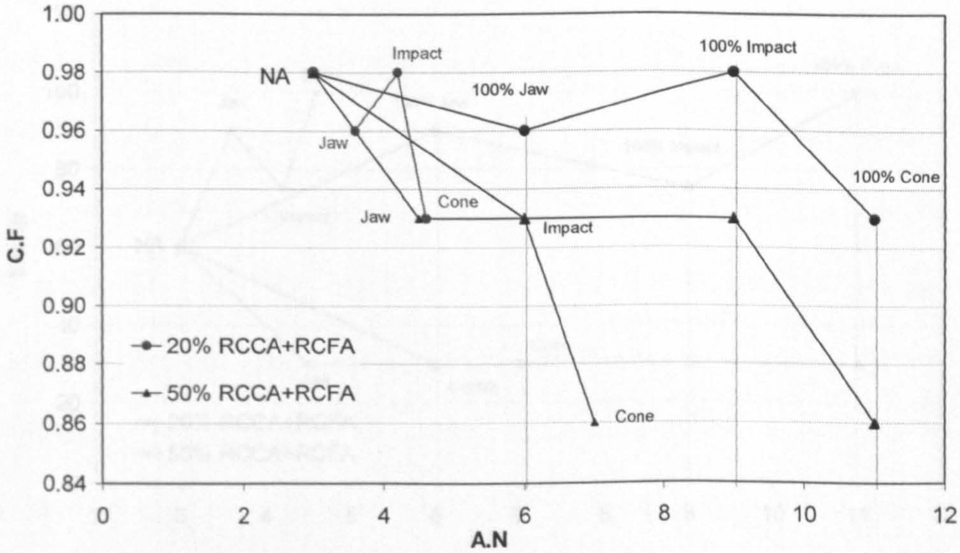


Figure 5-6: Effects of aggregates angularity number on CF for concrete with proportions of RCCA and RCFA (A.N values for 20% and 50% are derived linearly from 100% laboratory tested value)

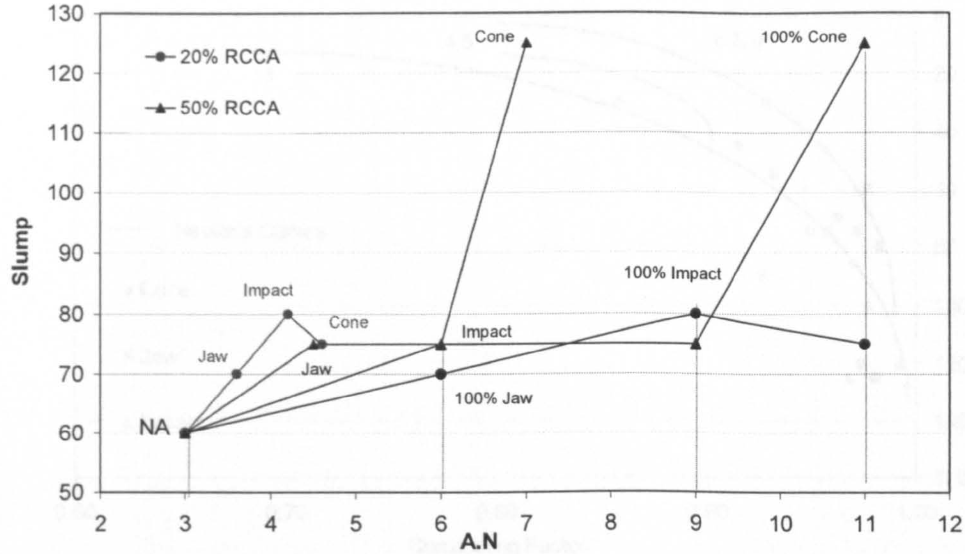


Figure 5-7: Effects of aggregates angularity number on slump for concrete with proportions of RCCA (A.N values for 20% and 50% are derived linearly from 100% laboratory tested value)

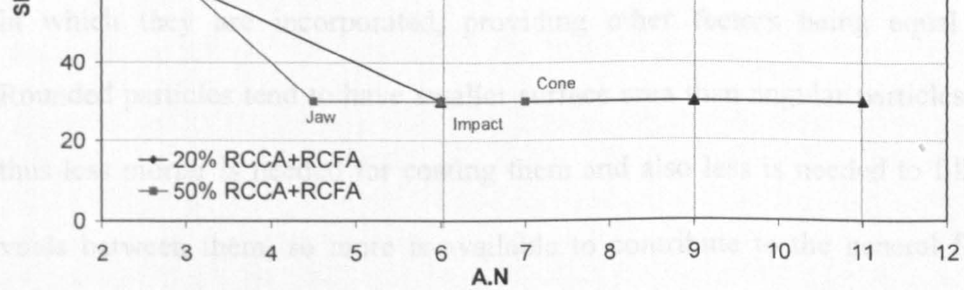


Figure 5-8: Effects of aggregates angularity number on slump for concrete with proportions of RCCA and RCFA (A.N values for 20% and 50% are derived linearly from 100% laboratory tested value)

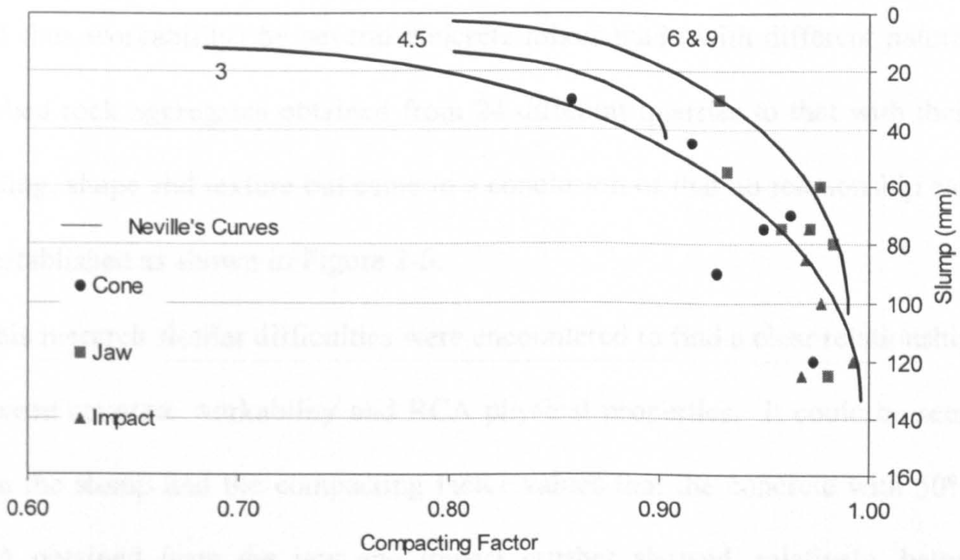


Figure 5-9: Slump - Compacting factor (Agg. / Cement = 4.5) for variable crushers with Neville's curves for mixes of varying agg. / cement ratio [59].

5.3 Discussion on Workability

There is a general belief that the shape of aggregate has considerable effect on the workability of concrete, it could be valid to say that the more nearly spherical are the particles of the aggregates, the more workable will be the mix in which they are incorporated, providing other factors being equal [68]. Rounded particles tend to have smaller surface area than angular particles and thus less mortar is needed for coating them and also less is needed to fill the voids between them, so more is available to contribute to the general flow-ability of the mix [68]. However, in practice there are other properties of aggregates that will alter any relationship between singular characteristics of aggregates with the workability.

This was reported in a comprehensive experimental study completed by BRE [55] where they have attempted to compare the amount of extra water required (and thus workability) by several concrete mixes made with different natural crushed rock aggregates obtained from 24 different quarries to that with their grading, shape and texture but came to a conclusion of that no relationship can be established as shown in Figure 2-6.

In this research similar difficulties were encountered to find a clear relationship between concrete workability and RCA physical properties. It could be seen from the slump and the compacting factor values that the concrete with 50% RCA obtained from the jaw and impact crusher showed, relatively, better workability than that from the cone crusher. At 20% RCA the value are too close to draw conclusions. This could be related to the fact that the jaw crusher produced RCA that was less flaky and angular than from the cone crusher, see Figure 5-5.

It was experienced that a 20% replacement of RCCA or RCFA added separately did not deteriorate the workability. It either remained close to or increased relative to the control mix. However, if both RCCA and RCFA were added together, the workability mostly reduced, from the 20% to 50% replacements (except for impact RCFA). If the replacement increased to 50%, concrete with RCCA showed that there is no significant difference compared to the control mix, while for RCFA a slight reduction was reported for slump values and a higher reduction in CF values. Compacting factor methods showed that the workability either remained similar to standard concrete or reduced. This differs from the slump values, which shows a distinctive increase if 20% of RCFA were replaced.

The effect of grading, in particular the low fraction passing the 600 μm sieve, would suggest a reduction in CF as the replacement RCFA increases. A change in CF from 0.98 to 0.93 is quite considerable (as the normal range for CF is 0.7 to 1.0) and could be interpreted as a three fold increase in the mixing air content (i.e. [1-0.93] / [1-0.98] is 3 times increase). The results for the RCCA are more encouraging with little change in the CF, with the exception of the cone crushed RCCA that we have already noted as being rather angular. The implications for compacting concrete are therefore less onerous (but still important).

5.4 Conclusion on Workability

The workability is very important to the commercial production of extruded or slip-formed hollow core floor units. Manufactures are careful to control workability by controlling water content, allowing the strength of concrete to fluctuate if the workability has to be adjusted.

Tattersall [68] reported that there are many factors affecting the workability and the situation is complicated further by the fact that there are an interaction between them and that they are not independent of each other in their effects this was also demonstrated by BRE [55] research work, see Figure 2-6.

The slump value of fresh concrete made with RCA varied widely depending on the percentage and type of replacement; a fact which may be linked to the angularity of the RCCA. The compacting factor of fresh concrete made with RCA was more consistent, and showed the problems encountered with using angular RCCA produced by the cone crusher, see Figure 5-5 and Figure 5-6. Replacement up to 50% of RCCA did not significantly affect the workability.

However, using up to 20% of RCFA causes the workability to increase and this possibly related to the extra water was not fully absorbed.

Combining the RCFA and RCCA will reduced the workability and the reduction will increase with higher replacement percentages, see Figure 5-1 & Figure 5-6. Clearly introducing RCFA to concrete mixes cause deterioration to the workability; similar findings were reported by BRE [55] on crushed fined rock aggregate where they found the concrete is less workable in comparable to gravel river sand. They have concluded that the amount of fine crushed aggregate could be adjusted to obtain the required concrete workability.

The Slump and CF patterns are fairly consistent in reporting the workability of concrete made with RCA. It is found that for a fixed recycled aggregate/cement ratio of 4.5 the slump decreases as the compact factor decreases, this agrees with Neville [59] as shown in Figure 5-9.

6 Strength & Density of Concrete with RCA

Introduction

This chapter covers the mechanical properties of concrete with the recycled aggregate obtained from the three different crushers and compares the result with that of the control mix. The tests are compressive cube strength, tensile splitting strength and flexural strengths. The tests were carried out on 3 days, 7 days and 28 days. This was to demonstrate the gain strength in standard concrete and that with RCA and to conclude any effects evolved from RCA replacements.

6.1 Hardened Concrete Properties

6.1.1 Compressive Strength

100 mm cubes were used to measure the compressive strength according to BS 1881, Part 116, 1983 [69]. The target compressive cube strength for the control mix was 60 N/mm^2. From each mix three cubes were tested at 3, 7 and 28 days (see mix design and W/C ratio in Table 5-2 to 5-4). The variance of the results are fairly consistent where all reported a ratio of standard deviation-to-mean to be less than 15%, see Appendix 2 for details.

Figure 6-1 shows the strength gain of concrete made with RCCA of 20% and 50% replacements derived from the cone, impact and jaw crushers with a 0.5 W/C design value, see section 5.1 for the standard mix design. It can be seen that all have similar trend behaviour where at 7 days they gained around 60% of the (28 days) target strength and 80% at 14 days, all within acceptable variation range to that of the control mix. Initial results appears to be encouraging as it shows proportionate usage (20% to 50%) of RCA derived from hcu in new high strength concrete may not have detrimental effect on its compressive strength gain. Considering the result in further details, the cone crusher 3 and 7 days strengths with 20% and 50% RCCA almost match the strength for control concrete, which was around 43 N/mm^2 and 53 N/mm^2, respectively. However at 28 days both with 20% and 50% cone RCCA the strength reached almost 67 N/mm^2, about 8% higher than the standard mix strength of 62 N/mm^2. Hansen and Narud [12] found that the compressive strength of RCCA concrete made with RCCA derived from high strength concrete is almost the same (and in some cases is higher) than the conventional concrete. This depends on the strength of 'original' concrete from where the RCCA is derived. If the water-cement ratio of the original concrete is the same as or lower than that of concrete made with these RCCA, then the strength of the concrete made with RCCA can be as good as or higher than the strength of the original concrete. This was also confirmed by Yoda *et al* [70] who found an 8.5% increase for concrete with RCCA compared to conventional concrete with same free W/C.

Concrete with RCCA from the jaw crusher is relatively lower than the control mix, especially for 20% replacement. At 3 days it reached almost 39 N/mm^2 (9% lower), at 7 days 46 N/mm^2 (13% lower`) and at 28 days 58 N/mm^2 (6% lower). However, it almost matched the standard concrete strength when the jaw RCCA replacements increased to 50%. For RCCA derived from the impact crusher, the strength behaviour did not differ significantly.

At 20% replacements and up to 7 days the strength gain was slightly lower than the standard mix. At 28 days the strength almost matched the control mix.

It could be seen that concrete with RCCA derived from both the impact and cone crusher shows relatively similar strength behaviour, and slightly high strength gain than that derived from jaw crusher. This coincides with what had been reported in section 4.1 that some physical properties for impact crusher aggregates are slightly better than the jaw aggregates; aggregate from the jaw crusher were reported to have higher flakiness index (15) and lower angularity number (6) compared to aggregates from the impact crusher which have 9 and 9, respectively.

Higher flakiness index causes more entrapment of water and air voids forming weaker internal layers that could relatively weaken the concrete strength. On the other hand less angular aggregates could result in less bonding and this could cause reduction in the concrete strength. However these properties could not be considered as the only driving factors that affect the concrete performance. For instance the water absorption, which is another crucial factor; the RCCA derived from the cone crusher showed lower water absorption than the RCCA jaw, but it is still reported to marginally have higher compressive strength than that of concrete with RCCA jaw.

Clearly there are several factors affecting the strength of concrete and to conclude a clear relationship between single properties of recycled aggregate to that of the strength concrete performance may not be possible to achieve. This was also demonstrated by BRE [55] on concrete made with natural crushed rock aggregates obtained from 24 different quarries all over the UK. Based on 550 different concrete mixes they came to a conclusion that the difference in the performance of concrete could not be related to any single characteristics of the aggregates. They reported that the compressive strength of concrete depends on the aggregates used but this cannot be related to any single physical characteristic of the aggregates. As an example to their conclusion Figure 2-5 shows ten percent fine values against compressive strength of concrete. The best-fit straight lines indicate no significant effect of aggregate strength on concrete compressive strength.

For RCFA replacements similar findings to that of RCCA were reported, see Figure 6-2. Concrete with 20% RCFA, which was derived from both the cone and impact crushers for up to 7 days, showed 10% lower gain in strength than the control mix, but equal strength at 28 days. When the RCFA replacements increased to 50% for both the cone and impact aggregates the strength gain up to 28 days was similar (and slightly higher) than the control mix. For RCFA derived from the jaw, the gain in strength reported to be the lowest up to 28 days, being 20% lower than the control mix for both 20% and 50% replacements. The water absorption did not differ much from that of the RCFA cone, which showed higher strength. It confirms the fact that beside water absorptions there are other combined factors affects the strength gain.

When joining RCCA and RCFA, Figure 6-3, all mixes showed similar trend behaviour to that of the control mix. However, Hansen and Marga [22] concluded that the use of 100% of both RCCA and RCFA could reduce the compressive strength by approximately 30% compared to concrete with natural coarse and fine aggregate; no details about the source of their RCA. They also found that the RCFA has a deteriorating effect on the compressive strength of recycled aggregate concrete. Ravindrarajah and Tam [31] reported similar results and explained that the detrimental effect of using RCFA in concrete could be eliminated by a partial replacement with natural sand. This coincides with our findings that replacement of RCFA from 20% up to 50% would have negligible effects on compressive strength gains.

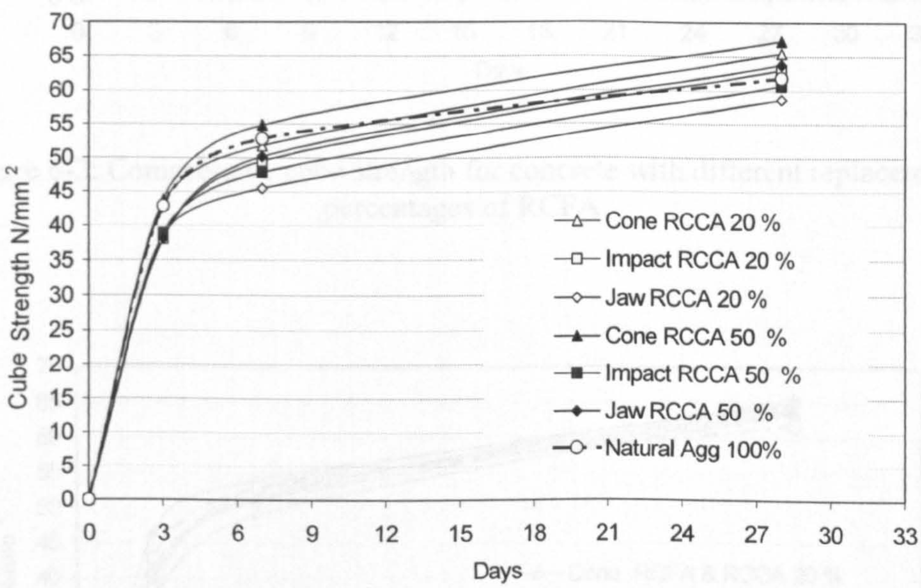


Figure 6-1 Compressive cubes strength for concrete with different replacement percentages of RCCA

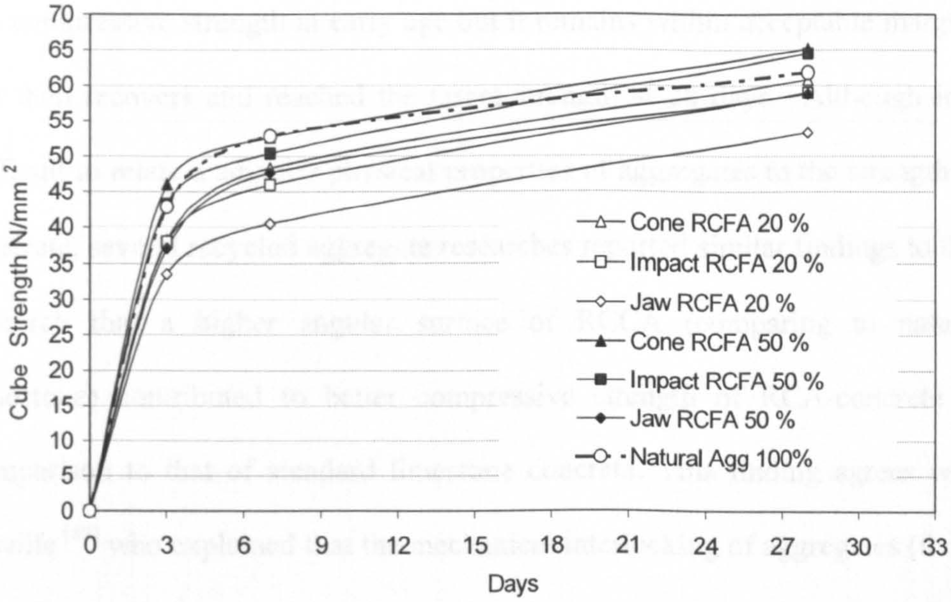


Figure 6-2: Compressive cube strength for concrete with different replacement percentages of RCFA

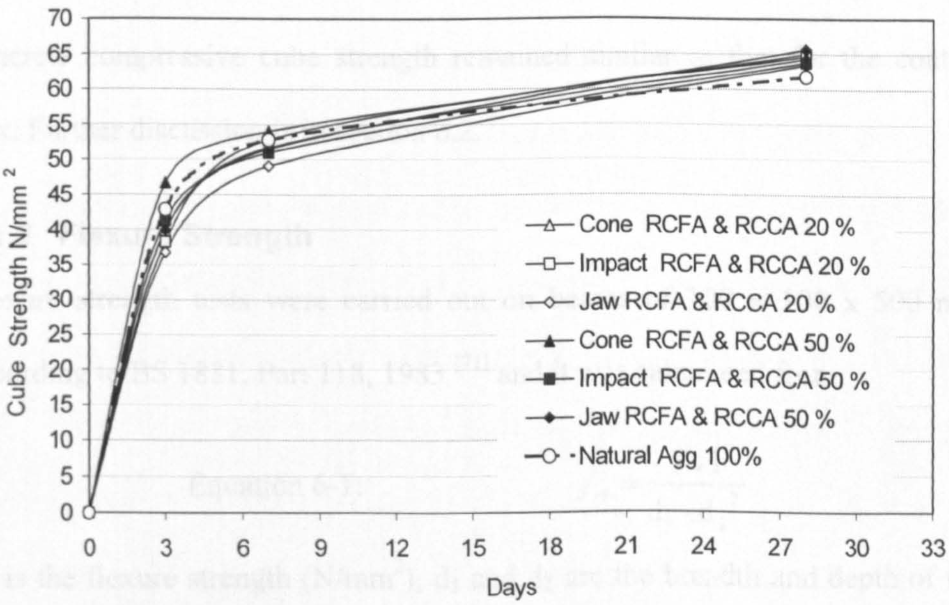


Figure 6-3: Compressive cube strength for concrete with different replacement percentages of RCCA & RCFA

In conclusion concrete with RCCA shows better compressive strength behaviour than the control mix while concrete with RCFA relatively reduced the compressive strength at early age but it remains within acceptable margins but then recovers and reached the target strength at 28 days. Although it is difficult to relate a singular physical properties of aggregates to the strength of concrete, several recycled aggregate researches reported similar findings to this research that a higher angular surface of RCCA (comparing to natural limestone) contributed to better compressive strength of RCA-concrete in comparison to that of standard limestone concrete. This finding agrees with Neville [59] who explained that the mechanical interlocking of aggregates (those with rougher texture) do contribute to the strength of concrete in compression through better bonding with the cement paste. It also agrees with Guineaa [44] who concluded that compressive strength increases with angular crushed aggregates. Added to that, when combining RCCA and RCFA together the concrete compressive cube strength remained similar to that for the control mix. Further discussion is in section 6.2.

6.1.2 Flexure Strength

Flexure strength tests were carried out on beams of 100 x 100 x 500 mm according to BS 1881, Part 118, 1983 [71] and it was calculated from:

Equation 6-1: $$f_{ct} = \frac{F \times l}{d_1 \times d_2^{\,2}}$$

f_{ct} is the flexure strength (N/mm^2), d_1 and d_2 are the breadth and depth of the cross section of the beam in (mm), F is the breaking load in (N) and l is the distance between the supporting rollers in (mm). The variance of the results are fairly consistent where all reported a ratio of standard deviation to mean to be less than 15%, see Appendix 2 for details.

Concrete with all proportions of RCA generally showed 15% to 20% higher flexure strength than that of the control concrete mix at 28 days, see Figure 6-4 to Figure 6-6. The strength also increased when the replacement percentages increased, in a case up to 35% higher than the control concrete if both RCFA and RCCA from cone crusher were combined.

Concerning the effects of the crushing machine, no significant differences were reported between the aggregates from the various crushers. All showed almost similar behaviour. DETR report [45] concluded similar findings where that the flexural strength for concrete with RCA derived from precast concrete did not show any significant difference to the controls. Kawamura and Torii [72] also reported that the flexure strength of concrete made with RCCA was higher than when using conventional concrete.

In the flexural strength, the bond between aggregates and cement paste is an important factor. Figure 6-7 shows the fracture surface of tested flexural prism made from concrete with 100% natural aggregates. In a good bond the fracture surface should contains some aggregate particles broken right through and several more ones pulled out from their socket, this is roughly shown in Figure 6-7. However, if more fractured aggregates particles emerged then this suggests that the strength of aggregates is nearly close to the strength of cement paste. This could explain the fracture surface behaviour of the flexural prism with 50% RCCA and RCFA where there was a substantial amount of fractured RCA particles, see Figure 6-8.

This also agrees with Kawamura and Torii[72] findings where higher flexure fatigue strength in RCA concrete is due to the strong bond between cement mortar matrix and recycled aggregate particles. Other researches like Malhotra [73] and Ikeda [74] reported lower flexure strength for RCA concrete, but no details were reported as to the origin of the RCA used, but Hansen [6] refers these differences in conclusions to the differences in the quality of the recycled aggregate.

It was explained in Section 5.1 that it was aimed for the mix design to reach a target strength similar to the strength for the parent of the recycled aggregates. Clearly, these rougher textured recycled aggregates have bonded well with the new cement paste and the flexural failure propagated through these aggregates and caused higher value of tensile strength. Similarly, for tensile splitting strength, which is discussed in next section.

The correlation between compressive strength and tensile strength is of quite importance in the prestressing of concrete. Unlike conventional concrete, the prestressed concrete is more likely to be designed to exploit the tensile capacity of concrete to its allowable limits. BS 8110, Part 1, 1997[75] limits the relationship between the design flexural strength and the compressive strength for class 2 pretensioned beams to:

Equation 6-2: $$f_{ct} = 0.45\sqrt{f_{cu}}$$

f_{ct} is the flexure strength (N/mm^2), f_{cu} is the compressive strength (N/mm^2).

Table 6-1 shows that the constant K for the concrete with RCA (all types) is more than 1½ times the value of K = 0.45 suggested by BS 8110.

The value for the control concrete is reported to have a lower K value than the concrete with different type of RCA. This provides a justification that the recommended relationship between f_{ct} and f_{cu} is valid for mixes using natural aggregates and those replaced with certain proportions of RCA.

It was as expected that the flexural strength was higher than the tensile splitting strength. In tensile splitting strength (discussed in next section) the concrete is subjected to tensile and compressive stresses in perpendicular directions and the strains are additive, causing a reduction in the tensile splitting strength. However, in the flexural test, there is no stress at right angles to the tension which helps to increase its values.

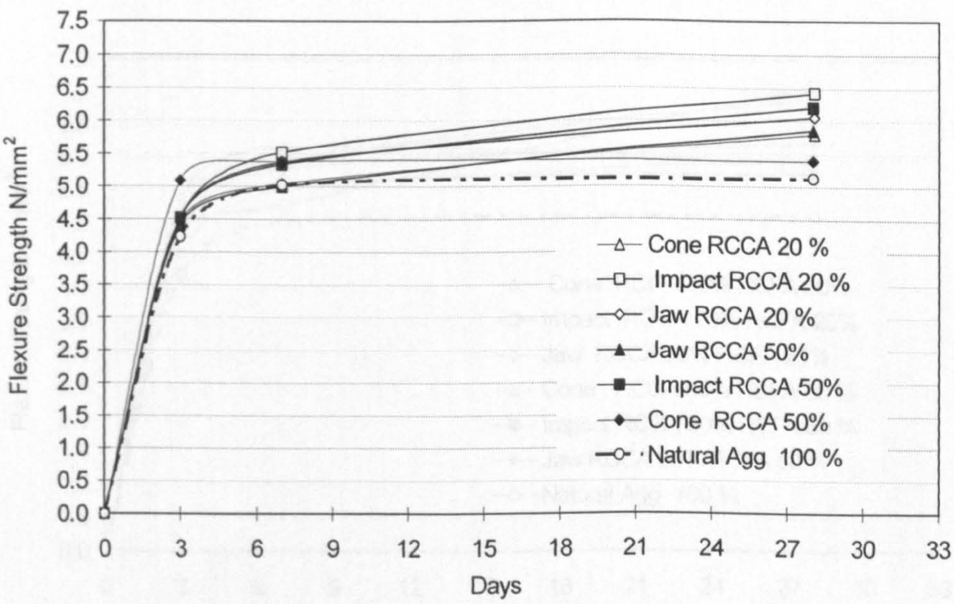


Figure 6-4: Flexure strength for concrete with different replacement percentages of RCCA

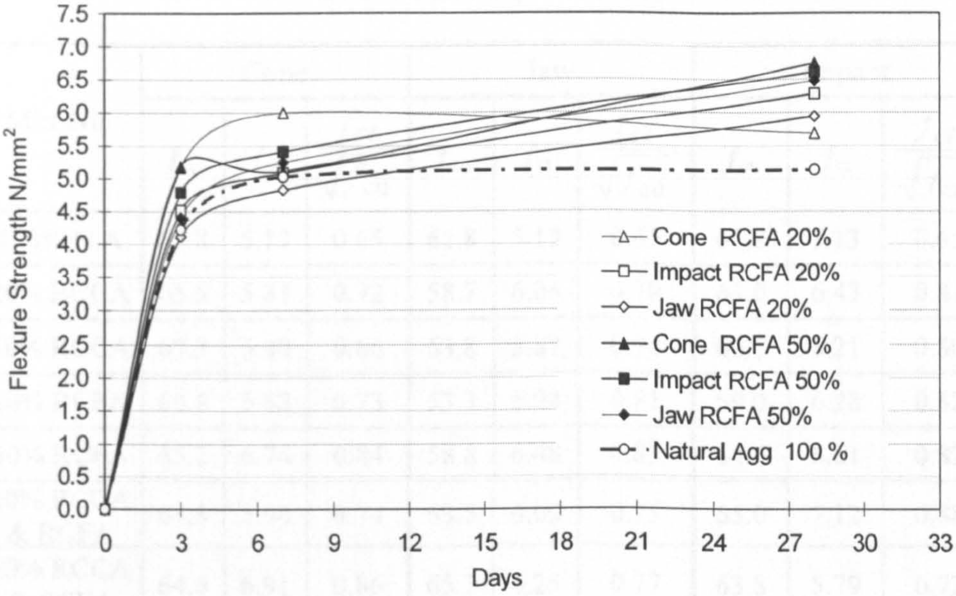


Figure 6-5: Flexure strength for concrete with different replacement percentages of RCFA

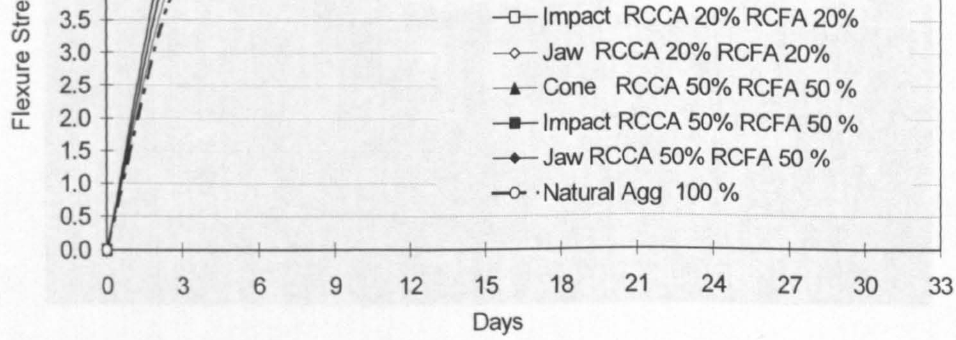


Figure 6-6: Flexure strength for concrete with different replacement percentages of RCCA and RCFA

Table 6-1: The relation between the compressive and the flexural strength according to Equation 6-2

Mix No.	Cone			Jaw			Impact		
	f_{cu}	f_{ct}	$\frac{f_{ct}}{\sqrt{f_{cu}}}$	f_{cu}	f_{ct}	$\frac{f_{ct}}{\sqrt{f_{cu}}}$	f_{cu}	f_{ct}	$\frac{f_{ct}}{\sqrt{f_{cu}}}$
100% N.A	61.8	5.13	0.65	61.8	5.13	0.65	61.8	5.13	0.65
20% RCCA	65.5	5.81	0.72	58.7	6.06	0.79	63.0	6.43	0.81
50% RCCA	67.3	5.40	0.66	63.8	5.87	0.74	60.7	6.21	0.80
20% RCFA	60.8	5.68	0.73	53.3	5.94	0.81	59.0	6.28	0.82
50% RCFA	65.2	6.74	0.84	58.8	6.48	0.85	64.5	6.61	0.82
20% RCCA & RCFA	64.3	5.94	0.74	63.5	6.00	0.75	65.0	7.12	0.88
50% RCCA & RCFA	64.6	6.91	0.86	65.7	6.25	0.77	63.8	5.79	0.72
Average	64.6	6.08	0.76	60.6	6.10	0.78	62.7	6.41	0.81

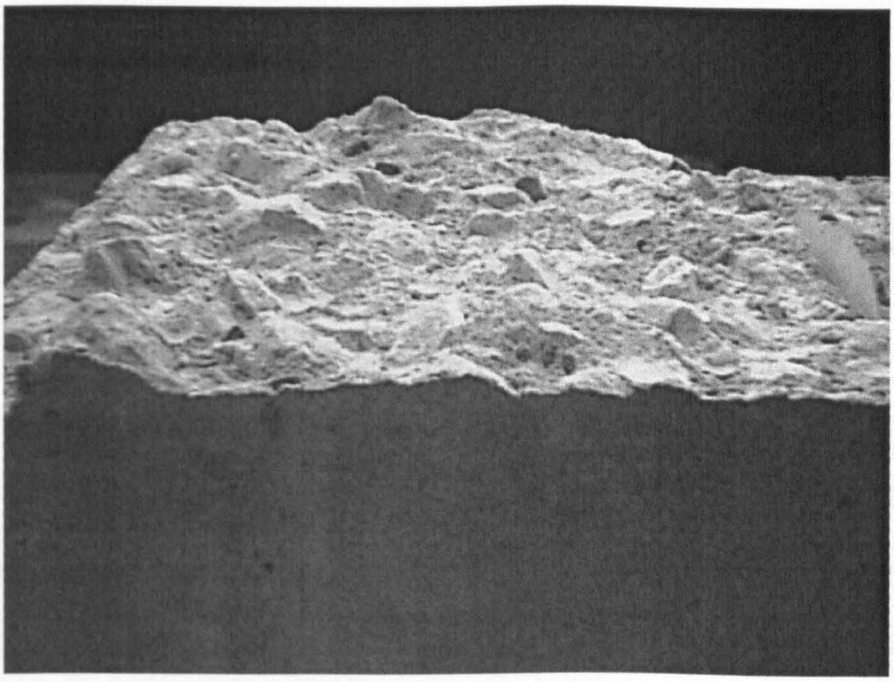


Figure 6-7: Fracture surface of tested flexural prism for concrete with 100% natural aggregates

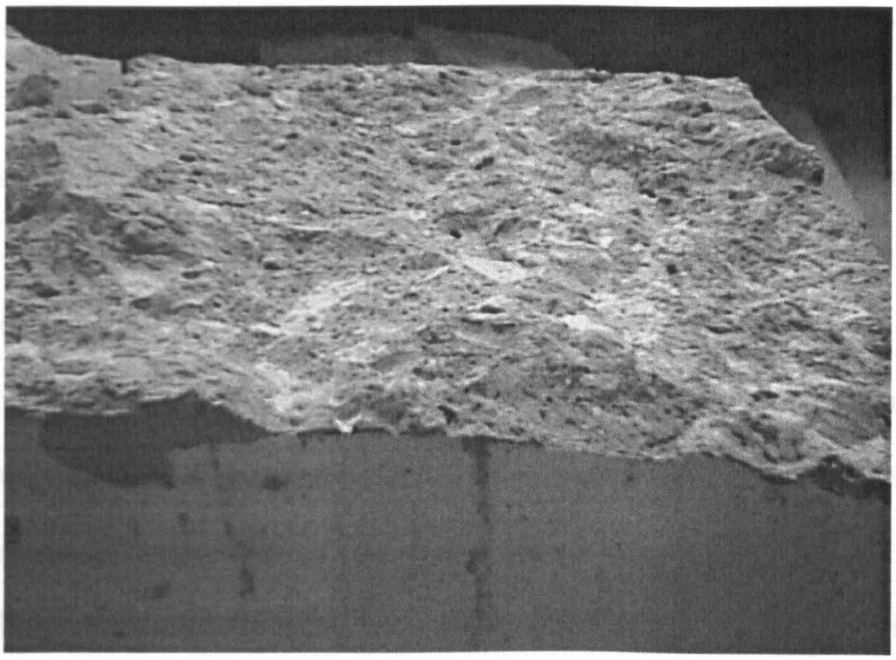


Figure 6-8: Fracture surface of tested flexural prism for concrete with 50% RCCA & RCFA

6.1.3 Tensile Splitting Strength

Tensile splitting strength was carried out according to BS, 1881, Part 117, 1983[76]. Cylinders of 150 mm x 150 mm were used and the Tensile splitting strength (N/mm^2) was calculated from:

Equation 6-3: $$f_{ts} = \frac{2\,\mathrm{F}}{\pi \times l \times d}$$

F is the failure load (N), l is the cylinder length (mm) and d is the cylinder diameter (mm). The variance of the results are fairly consistent where all reported a ratio of standard deviation to mean to be less than 15%, see Appendix 2 for details.

The tensile splitting strength for concrete with 20% replacements of RCCA derived from the impact crusher shows a relatively similar trend in behaviour to that of the control concrete, see Figure 6-9. When the replacements of RCCA impact increased to 50%, the strength gain showed higher values than the control mix at 28 days.

Both replacements of 20% and 50% of RCFA that derived from impact crusher, Figure 6-10, showed no deterioration effects on the tensile splitting strength and in some cases showed slightly higher values than the control mix. Similar findings were reported when joining both RCCA & RCFA that derived from impact crusher, see Figure 6-11. These could be related to the rougher texture of RCA that leads to larger attachment force to the cement matrix. Similarly, the large surface area of angular RCA means a larger bonding force can be developed.

No distinctive difference was reported for both the cone and jaw crushers. Figure 6-9 to Figure 6-11 show that the strength gain is within a close range. This agrees with Hansen [6] who indicated that B.C.S.J. [15] and Ravindrarajah and Tam [31] reported that there is no significant difference in the tensile splitting strength between concrete with RCCA and that with conventional concrete. Similarly Coquillat [77] reported that there is no significant difference compared to conventional concrete even if both RCCA and RCFA were used.

BS 8110, Part 1, 1997 [75] suggests the following relationship between the design principle tensile strength and compressive strength as follows:

Equation 6-4: $$f_{ts} = 0.24 \times \sqrt{f_{cu}}$$

Where f_{ts} is the tensile splitting strength, f_{cu} is the compressive strength and both are in N/mm^2. The K value of 0.24 includes a factor of safety of 1.5 on f_{cu} according to BS 8110. However, the K value could be increased to 0.3 based on the actual f_{cu} without the safety factor [1]. Table 6-2 shows the K value for concrete with RCA (almost all types) is nearly twice the value of that in Equation 6-4 and about similar to the value for the control mix. This also shows that the relationship between f_{ts} and f_{cu} is valid for mixes using natural aggregates and those replaced with certain proportions of RCA.

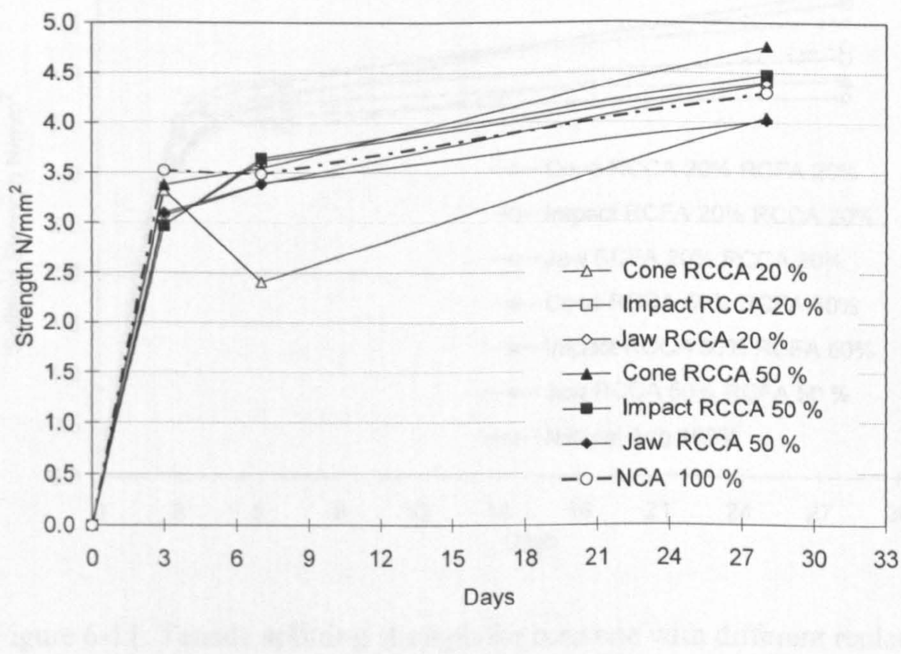


Figure 6-9: Tensile splitting strength for concrete with different replacement percentages of RCCA

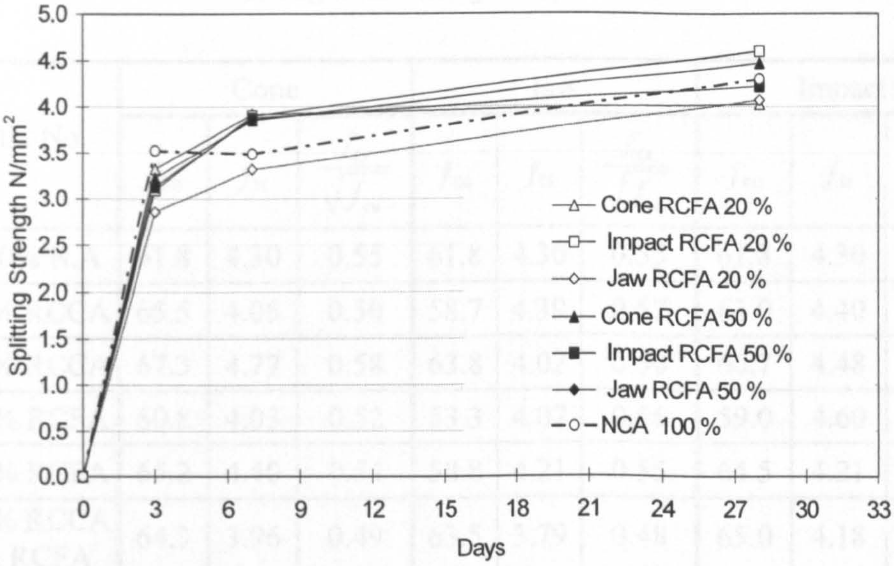


Figure 6-10: Tensile splitting strength for concrete with different replacement percentages of RCFA

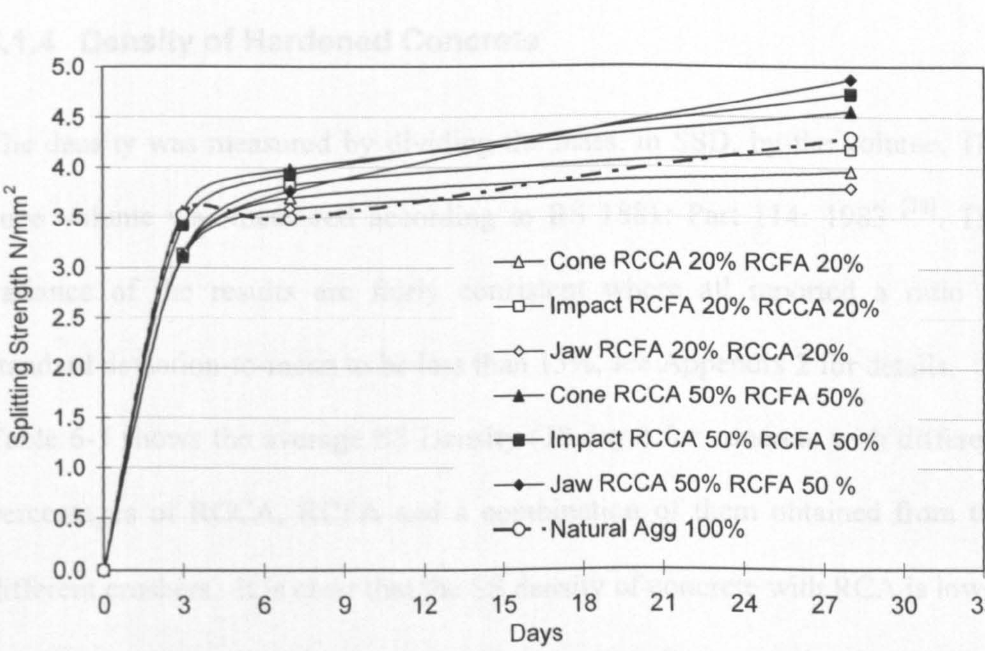


Figure 6-11: Tensile splitting strength for concrete with different replacement percentages of RCCA and RCFA

Table 6-2: The relation between the compressive and the tensile splitting strength according to Equation 6-4

Mix No.	Cone			Jaw			Impact		
	f_{cu}	f_{ts}	$\frac{f_{ts}}{\sqrt{f_{cu}}}$	f_{cu}	f_{ts}	$\frac{f_{ts}}{\sqrt{f_{cu}}}$	f_{cu}	f_{ts}	$\frac{f_{ts}}{\sqrt{f_{cu}}}$
100% N.A	61.8	4.30	0.55	61.8	4.30	0.55	61.8	4.30	0.55
20% RCCA	65.5	4.05	0.50	58.7	4.39	0.57	63.0	4.40	0.55
50% RCCA	67.3	4.77	0.58	63.8	4.02	0.50	60.7	4.48	0.57
20% RCFA	60.8	4.03	0.52	53.3	4.07	0.56	59.0	4.60	0.60
50% RCFA	65.2	4.40	0.54	58.8	4.21	0.55	64.5	4.21	0.52
20% RCCA & RCFA	64.3	3.96	0.49	63.5	3.79	0.48	65.0	4.18	0.52
50% RCCA & RCFA	64.6	4.55	0.57	65.7	4.87	0.60	63.8	4.72	0.59
Average $_{RCA}$	64.6	4.29	0.53	60.6	4.23	0.54	62.7	4.43	0.56

6.1.4 Density of Hardened Concrete

The density was measured by dividing the mass, in SSD, by the volume. The cube volume was measured according to BS 1881: Part 114: 1983 [78]. The variance of the results are fairly consistent where all reported a ratio of standard deviation-to-mean to be less than 15%, see Appendix 2 for details.

Table 6-3 shows the average SS Density (28 days) for concrete with different percentages of RCCA, RCFA and a combination of them obtained from the different crushers. It is clear that the SS density of concrete with RCA is lower than that of the control concrete and is getting lower as the replacement percentages increases, see Figure 6-12 for concrete with RCCA and Figure 6-13 for RCFA.

A greater reduction is found when RCCA and RCFA are combined and is even greater when the replacements increased to 50%, see Figure 6-14.

This was expected because the density of concrete normally is an arbitration between the density of the aggregates and the density of the cement mortar subjected to how well the concrete is compacted i.e. the existence of voids. The density of crushed limestone aggregates is around 2600 kg/m^3 while that of cement paste is estimated around an average of 2000 kg/m^3 and since the concrete is a non-homogeneous material consists of hydrated cement paste and aggregate; the lower density of cement paste cause the density of concrete to drop to around 2400 kg/m^3. Concrete with RCA is expected to have more proportions of cement mortar with lower density values to that of limestone. The higher proportions of RCA the more the quantity of old cement paste the lower the density of concrete with RCA will be.

There are other factors that could contribute to high reductions in density; these factors were experienced in the laboratory while casting and preparing the specimens. One of the main factors was the bleed water due to the recycled aggregates, which presumed to being unable to hold the water when they settled downwards; however there was no measurement for the bleeding as it was out of the research scope. Bleeding cause miniature channel/voids, which contribute to lower density.

The lower density findings is in line with Hansen [6] who reported that the density of recycled aggregate concrete is always lower than that of control concrete but no specific values were mentioned. Concerning the crushing effects it was found that the SS densities for concrete with RCA derived from jaw crusher are slightly lower than that of impact and cone crushers. Concrete with cone RCA have higher densities than the other two although the cone RCCA has a higher flakiness index and more angularity than from the other crushers.

Table 6-3: Surface saturated density kg/m^3 (28 days) of concrete with recycled aggregate of different sources and percentages (average)

Replacements	Cone	Jaw	Impact
0% Control Mix	2418	2418	2418
20% RCCA	2412	2390	2405
50% RCCA	2388	2393	2382
20% RCFA	2423	2400	2395
50% RCFA	2405	2385	2392
20% RCCA+ 20% RCFA	2412	2400	2395
50% RCCA+ 50% RCFA	2362	2365	2370

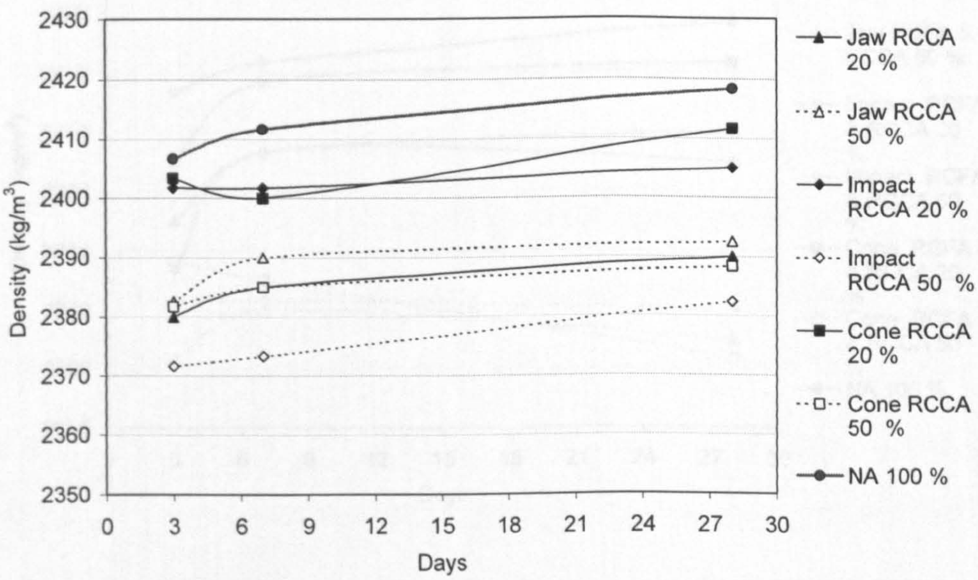


Figure 6-12: Surface saturated density for concrete with RCCA derived from different crushers

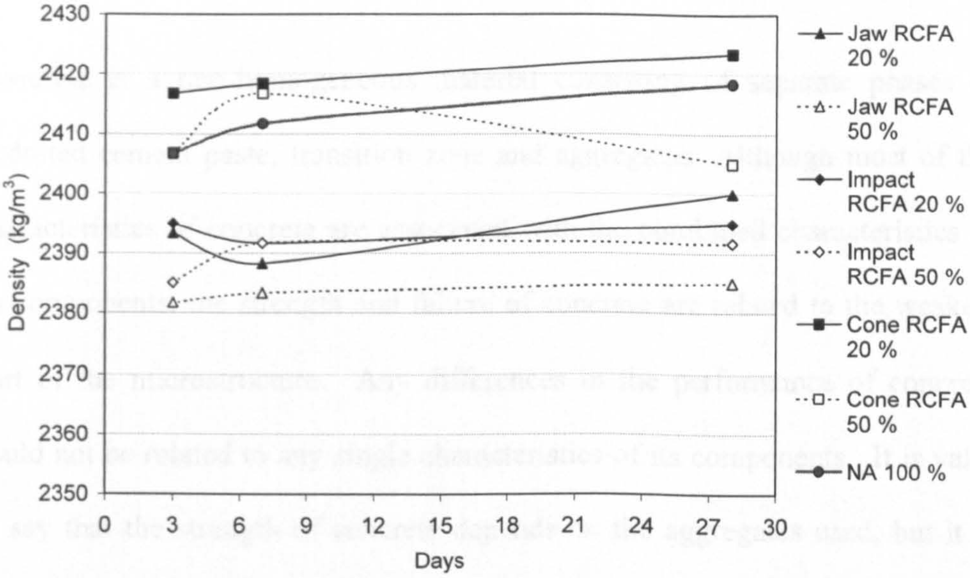


Figure 6-13: Surface saturated density for concrete with RCFA derived from different crushers

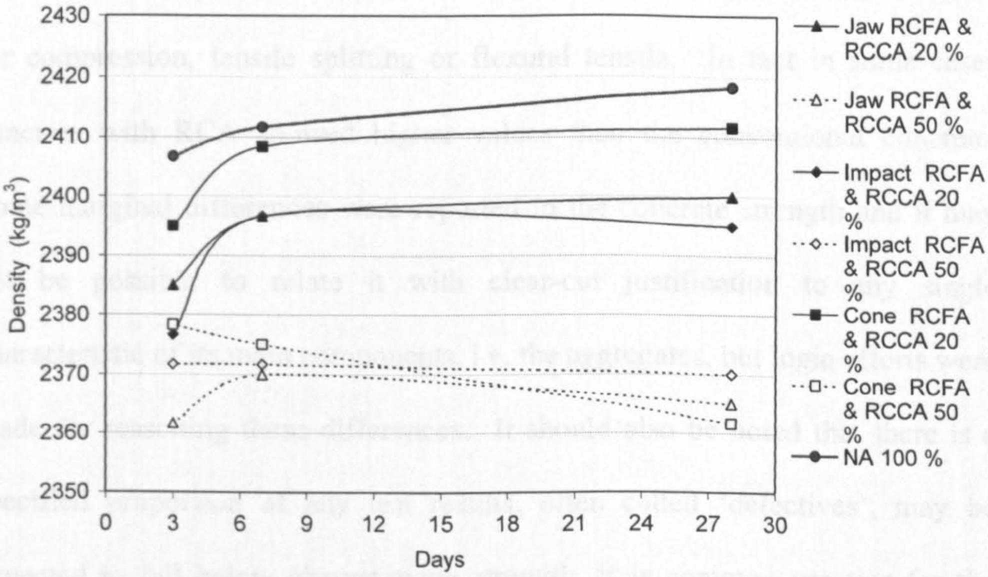


Figure 6-14: Surface saturated density for concrete with RCCA & RCFA derived from different crushers

6.2 Discussion

Concrete is a non-homogeneous material consisting of separate phases of hydrated cement paste, transition zone and aggregates. Although most of the characteristics of concrete are associated with the combined characteristics of its components, the strength and failure of concrete are related to the weakest part of the microstructure. Any differences in the performance of concrete could not be related to any single characteristics of its components. It is valid to say that the strength of concrete depends on the aggregates used, but it is equally correct to say such strength cannot be related to any single physical characteristic. The results of this research to this point showed that concrete with RCA derived from hollow core slab units with certain proportions up to 50%, did not cause any distinctive reduction on the strength of concrete either for compression, tensile splitting or flexural tensile. In fact in some cases concrete with RCA showed higher values than the conventional concrete. Some marginal differences were reported in the concrete strength and it may not be possible to relate it with clear-cut justification to any single characteristic of its main components, i.e. the aggregates, but logic efforts were made for reasoning these differences. It should also be noted that there is a specified proportion of any test results, often called 'defectives', may be expected to fall below characteristic strength. It is common practice for the characteristic strength to be defined to have a proportion of defectives; BS 5328 and BS 8110 adopt the 5% defective level in line with the CEB/FIP international recommendations for the design and construction of concrete structures.

The density of concrete is proven to be more sensitive to the components of concrete, including higher water absorption and the degree of compaction. The concrete density is reported to be lower than the conventional when the RCA was replaced and it tends to decrease more as the replacement increases; this is related to the increased amount of the cement paste attached to RCA, which is known to have a lower density.

The reduction in density for concrete with RCA was limited to around 2360 kg/m^3 (down from 2410 kg/m^3 for standard concrete), although higher amount of cement paste attached to RCA contributed to density reduction in concrete, but this reduction is halted to this level because the RCA are derived from hcu, which is a high strength high quality dense concrete. The results of the strength tests, compressive and flexural, shows that such reduction in density has limited effects on the strength gain of the concrete.

It is known that the cement paste contains pores, fissures and voids, which may influence the concrete density, but still not fully known by what mechanism they affect the strength gain. The voids themselves in general may not act as a defect [59]; the defects are likely to be cracks propagated due to different reasons that affect the bonding between the cement paste and the aggregates. Giving that the source of recycled aggregate is of high quality high strength hcu, defects are likely to be minimal as seen by a good strength gain for RCA-concrete in comparison to standard concrete. This also can be confirmed by Nagatakia [17] who found in a study that the adhered mortar is not always the primary parameter determining the quality of the recycled coarse aggregate; they found sandstone coarse aggregate originally had defects in the form of voids and cracks and added that the elimination of the friable and porous aggregate particles during recycling and crushing process created almost micro defect-free recycled coarse aggregates with a high level of integrity resulting in better mechanical performance.

When RCCA and RCFA are added separately, the results show that there is good performance in concrete compressive strength; it is in competition with that of natural aggregate. This suggests that certain proportions of RCA derived from high quality concrete are adequate and perform well in the production of similar strength concrete to that used in precast concrete production, see Figure 6-1, 6-2 and Figure 6-3.

Concrete with RCCA and RCFA separately, or with a blend of them, showed better performance in flexural strength than the conventional concrete. In most cases concrete with RCA showed higher tensile strength than that with natural limestone. The tensile strength value increases as the replacement percentage increased. Although the nature of the bond between the aggregates and cement paste is not entirely understood, but the general belief is that it is clearly influenced by the aggregate properties. A rougher and more angular surface will result in a better bonding due to the mechanical interlocking which in turn affects the tensile fracture surface. The failure of bonding should either go around the aggregate particle or break straight through them depending on the aggregate and the paste strength. This is also in line with Guineaa [44] who found that the strength of the interface between aggregate and the matrix affects the fracture energy in different ways depending on the shape of the particles and reported that concretes with crushed angular aggregates show a higher value of fracture energy and in consequence high compressive and tensile strength. The concrete fracture energy is defined as the energy absorbed to create a unit area of fracture surface.

This result could agree with the texture of RCA where it was reported to be more angular and rougher texture and this would give better interlocking, higher bonding and subsequently more resistance to flexure. As Figure 6-8 shows the fracture surface mostly went through the RCA particles. The higher gain strength could also be related to the gel/space ratio. The gel/space ratio is known as the ratio of the volume of the hydrated cement paste to the sum of the volume of the hydrated cement and the capillary pores. Neville [59] explained that cement hydrates occupy more than twice their original volume. The higher gel/space ratio the better bonding with aggregate and thus higher strength gain. It is likely that concrete with RCA provides adequate capillary pores that improve the gel/space ratio, which provides good bonding and subsequently a higher strength gain and could also help to maintain cement hydration over time which help increase the strength gain over time as well as static modulus of elasticity (E) and poisson's ratio (υ). These are fundamental parameters necessary in structural analysis for the determination of the strain distributions and displacements, especially when the design is based on elasticity considerations, as in the case of prestressed concrete including hollow core slab units. The values of E and υ of concrete depend on the values of E and υ of cement paste as well as to that of aggregates.

Recycled aggregate is a combination of natural aggregates and cement paste. Hydration of cement paste is a continuing process resulting in strength increase for concrete over time. Sideris [79] reported that current equations that relate of E and υ to that of concrete strength does not include the hydration period, and ignores these values at later ages. Although his research was performed on concrete with natural aggregates, this could be important for concrete with RCA where the attached cement paste may continue to hydrate. He reported that the cement hydration ends at the age of around 15 years, and by using the cement hydration equation investigated the respective individual relationships between compressive strength of concrete and E and υ. He found that the magnitude of the difference between of E and υ at 28 days and to that by final hydration of concrete ranging up to about 75% for modulus of elasticity and 32% for Poison ratio. This implies that the elastic characteristics of concrete significantly increase at final hydration in respect to that determined at 28 days, and for this reason their ultimate values must be taken in account by the prediction of the displacements in structures.

This was also supported by Kou and Poon [80] who carried out a long-term study on the mechanical properties of recycled aggregate concrete. He reported that compressive strength, tensile splitting strength and modulus of elasticity all have considerably high gain compared to that of natural aggregates, as follows

Table 6-4: Percentage Gain from 28 days to 5 years for compressive strength, tensile splitting strength and modulus of elasticity

% Gain from 28 days to 5 years			
RCA %	Compressive Strength: MPa	Tensile splitting strength: MPa	Modulus of Elasticity: GPa
0%	34	37	20
20%	40	40	23
50%	52	47	25
100%	53	57	36

Concerning the crushers' effect, all show similar trend behaviour in concrete strength, but some marginal differences are reported. RCA derived from the cone and the impact crushers performed marginally better in concrete strength to that from the jaw crusher. For flexural strength, the RCA from the impact crusher showed higher values, while RCA from the cone crusher has a higher concrete density. This is in contrast to the aggregate properties which, as discussed in the previous section, the aggregate derived from cone crusher are good quality aggregate but are lower quality than from the jaw and impact crushers. Nevertheless all aggregates reported to perform as good as natural aggregates in concrete strength.

6.3 Conclusion

In conclusion, concrete with RCCA from hollow core units show higher compressive cube strength than the control mix, while concrete with RCFA reduced the strength. This agrees with Hansen and Marga [22] and Ravindrarajah and Tam [31]. However, adding RCCA and RCFA together the compressive cube strength remained similar to that for the control mix.

Although the RCA from the jaw crusher performed sufficiently well in compressive strength, the impact and cone crusher show relatively a better performance.

For tensile strength, RCA showed better performance than that of natural limestone and even better, especially for flexure strength, if the replacement increased. These findings agree with DETR Report [45] and Kawamura [43]. Clearly, these recycled aggregates have bonded effectively with the new cement paste and the flexural failure just went through these aggregates and caused higher value for the tensile strength, see Figure 6-8. The relationship between f_{ct}, f_{ts} and f_{cu} in BS 8110, Part 1, 1997 [75] and between f_{ts} and f_{cu} are equally valid for mixes using natural aggregates replaced with certain proportions of RCA.

The SS density of concrete with RCA is lower than that of the control concrete and reduces further as the replacement percentages increases. This agrees with Hansen [6]. A higher reduction occurred when adding RCCA and RCFA together; this is because of concrete with RCCA and RCFA is expected to have more proportions of cement mortar, which is more porous and have lower density value to that of limestone-aggregate density. Other factors like the bleeding, which was clearly visible during tests may also diversely affected the density. However, strength test result showed that the limited reduction in density has no effects on the concrete strength.

Concerning the crushing effects all are found to produce acceptable recycled aggregate; but in comparison it was found that the SS densities for concrete with RCA derived from jaw crusher are marginally lower than that of impact and cone crusher.

Concrete with cone RCA has a higher density values than the other two although the cone RCCA showed a higher flakiness index and more angularity than the other crushers. The differences in aggregates characteristics from different crushes showed no significant difference in concrete strength and thus any of them could be used for production of recycled aggregates.

It was important to examine the mechanical properties, compressive and tensile strength, of recycled aggregates concrete because together with adhesion and friction are fundamental properties to the bond behaviour of prestressing wires in concrete; furthermore, the bond behaviour, which will be studied in the next chapter, is crucial for the development capacity of the precast prestressed concrete hollow core slab units.

7 Bonding Between RCA Concrete and Reinforcement

Introduction

Having studied the mechanical properties of concrete with RCA in previous chapters it is essential now to step up the study further to RCA concrete with reinforcement. Bonding between the concrete and reinforcement is what makes concrete reinforced and prestressed and therefore the bonding mechanism is an important requirement in demonstrating the ability of concrete to transfer tensile stress to the reinforcement. This section presents an investigation into the effect of using RCA on the bond between concrete and both reinforcing bars and prestressing wires. Both reinforcements were examined to encourage using RCA derived from hcu in precast prestressed concrete as well as ordinary concrete production. The experimental programme was carried out by tensioning the reinforcement placed centrally in prismatic concrete section, see Figure 7-3. The development of bond stress was deduced by measuring the distance between tension cracks. The tension versus elongation was also considered and discussed.

7.1 Principle of the Bond Test

Bond between pretensioning wire and the mortar fraction of concrete is a very important parameter concerning the development of precompression at the ends of prestressed units. This is particularly important to the shear and bearing capacity of precast concrete elements, such as hollow core slab units (hcu) that do not contain shear reinforcement. During manufactures of hollow core slab units (hcu), vibration of wire and strand results in a layer of cement and sand rich mortar at the surface. Profiles of strand left behind after debonding are totally bereft of coarse aggregates

Specific reference to the importance of bond in hcu's is given by Akesson [81] and Den Ujl [82] and has been particularly addressed in FIP 1986 [83] and FIB 2000 [84] documents. This is because the cover to pretensioning wire is small, e.g. 30 mm on three sides, as shown in Figure 7-1.

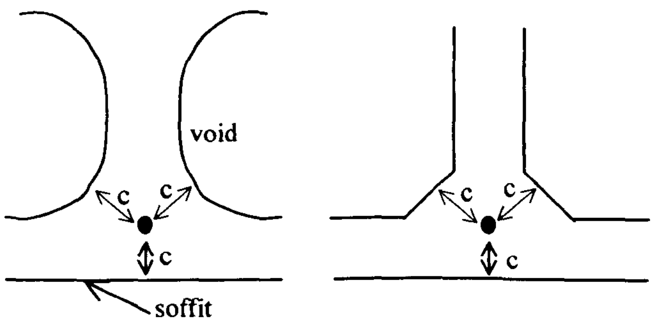


Figure 7-1: Cover to pretensioning wire in a section of a hollow core slab unit

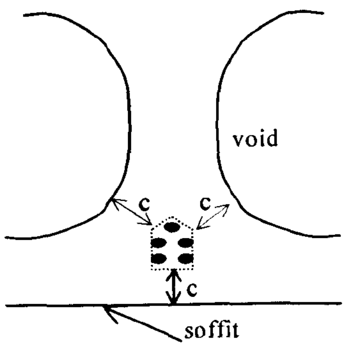


Figure 7-2: Cover to group of wires in a section of a hollow core slab unit

According to the FIP [83] the cover necessary to achieve full bond in fully stressed reinforcement should be approximately 2 diameters and the recommendation was based on the following equation:

Equation 7-1 $$\sigma_r < \frac{2\,c\,f_{ctk}}{d}\,,\quad f_{bd} = 0.4\,\sigma_r = 0.5\sqrt{f_{cu}}$$

Where σ_r is the radial stress, c is the concrete cover, f_{ctk} is the characteristic concrete tensile strength, d is the bar diameter, f_{bd} is the bonding strength and f_{cu} is the compressive strength. If a group of wires is used as a replacement for strand, Figure 7-2, the situation is even more critical because of the interaction between the numerous wires.

When prestressed hollow core slab units are manufactured by the shear compaction method, the compaction of concrete around the wires is controlled by the correct delivery of a cohesive concrete mix. Workability is carefully controlled by using "no slump" concrete that will compact into a ball by hand - the water/cement ratio is about 0.3. For good compaction at the bottom of the units the manufacturers of hcu's recommend that the size of the coarse aggregate should be 14 mm (or 10 mm depending on type of machine) down to a minimum size of 5mm. Its shape should be 'angular' to 'rounded' with an aspect ratio of about 1:2. The ratio of fine to coarse aggregate should be about 1 in 3. If these parameters are not controlled, as in the case of introduction of new materials as recycled concrete, information on the bond performance of these materials relative to the control must be determined. Full scale bond testing is therefore required. Figure 4-4 is a typical demonstration for the importance of mortar; it shows hcu with poor bond are linked to the quality of fine aggregates are not being controlled; see Table 7-2 for typical aggregates grading for UK hcu production.

The concern over bond stress has been heightened by the recent discovery of reduced shear capacity of hollow core slab units bearing onto flexible supports, e.g. Pajari [85]. The findings of Pajari at VTT [85] in Finland, which have been published by the FIB [84], show that curvature of the support beam induces a 3rd dimension of shear in the webs, thus increasing the principal tensile stress in the web. The shear capacity of hcu's is reduced by up to 44 % [85]. This effect is also manifest by longitudinal cracking - suggesting breakdown of bond at the bottom of the web.

As a simple, non scientific recommendation, the FIB documents suggests that 2 strands should be effectively considered as debonded in these circumstances. This recent work further justifies the concern for bond in this project.

Reproducing the geometry in Figure 7-1 in a bond test is clearly not practical, and would in any case lead to results that were geometry specific. A more practical solution would be a square section proportioned such that the cover to the mid faces was within the typical range of 30 to 40 mm. A prismatic section 75 x 75 mm is therefore suitable, as shown in Figure 7-3.

Bond pull out tests are notoriously difficult to perform if the resistance pressures to the pull out force are not to interfere with the bond, by laterally restraining the radial tension. Lorentsen [86] explains this point, and suggests a rather cumbersome test arrangement. Paine [87] (in this department) needed a concrete block weighing nearly 600 kg in order to correctly measure bond in 12.5 mm pretensioning strand.

However, because in this work we do not require an absolute measure of bond, rather the relative bond performance between recycled and natural aggregate concrete, a simpler test arrangement was used.

This composed of tensioning a single steel bar placed centrally in a prismatic concrete section and deducing the development of bond stress by measuring the distance between tension cracks as shown in Figure 7-3. In this context this test shall be referred to as the "prismatic bar bond test" (PBB). The principle of the test is shown in Figure 7-3. Although the PBB test does not detect the breakdown of bond via radial tension and pull-out displacement data, it does however, under nominally identical testing procedures, provide a measure of the development of bond. To interpret the results it is necessary to know the experimental value of tensile cracking stress f_{ts} of the concrete. This is given in Section 7.2 and in Table 7-3.

7.2 Reinforcing Bar Bond Tests

Initially the PBB test was carried out using high tensile reinforcing bar of 10 mm diameter. The length of the prism was calculated to permit at least 4 number of cracks to develop based on a bond length of about 8 to 9 mm times the diameter, calculated as follows:

Equation 7-2 $$L = \frac{P}{f_b \, \pi \, d}$$

P is the force in the reinforcement bar to cause a tensile crack and could be defined as $P = f_{ts} \times A_c$, where f_{ts} is the concrete tensile strength, f_b is the bond stress and A_c is the cross section area of the concrete prism.

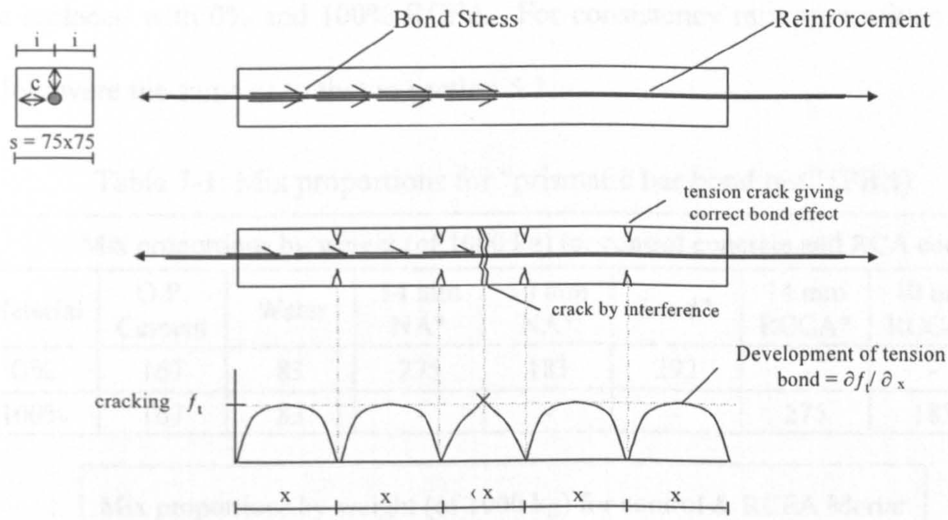


Figure 7-3: Prismatic section 75 x 75 mm used in a bond test with a diagram demonstrating concrete tensile stress at cracking point.

According to BS 8110: Part 1: 1997 [75], for high tensile deformed bar f_b is given as $f_b = 0.5 \times \sqrt{f_{cu}}$, and for normal concrete $f_{ts} = 0.24 \times \sqrt{f_{cu}}$. The development length is defined as follows [59]:

Equation 7-3 $$L = \frac{0.24\,A_c}{0.5\,\pi d} = \frac{0.48\,(s^2 - \pi d^2/4)}{\pi d}$$

where s = side dimension of the prism = 75 mm and d = bar diameter. For 10 mm diameter bar L = 85 mm, so to ensure that at least two cracks are formed at each end of the specimen, the total length of the prism must be 5 times the development length = 5 x 85 = 425 mm. Thus, a length of 500 mm is adequate, see Figure 7-4.

7.2.1 Mix Design

Two sets of tests have been used to study the PBB with reinforcing bar of 10 mm diameter. In set one, concrete mixes were used where both the fine and the coarse aggregates are replaced with RCCA and RCFA. The replacement percentages are

0% and 100%. In set two, only mortar mixes were used where the fine aggregates are replaced with 0% and 100% RCFA. For consistency mix proportions shown below were the same as to that in section 5.1.

Table 7-1: Mix proportions for "prismatic bar bond test" (PBB)

Mix proportions by weight (of 1000 kg) for control concrete and RCA concrete								
Material	O.P. Cement	Water	14 mm NA*	10 mm NA*	Sand*	14 mm RCCA*	10 mm RCCA*	RCFA*
0%	167	83	275	183	292	-	-	-
100%	167	83	-	-	-	275	183	292

Mix proportions by weight (of 1000 kg) for control & RCFA Mortar				
Material	O.P. Cement	Water	Sand*	RCFA*
0%	167	83	750	
100%	167	83		750

Table 7-2: Typical aggregates grading for UK hcu production

Typical fine aggregates grading for UK hcu production					
Sieve size	Sieve 0.3	Sieve 0.6	Sieve 1.18	Sieve 2.36	Sieve 5
% Passing	7	60	76	89	100

Typical course limestone grading for UK hcu production					
Sieve size	Sieve 20	Sieve 14	Sieve 10	Sieve 8	Sieve 6.3
% Passing	99	94	62	41	25

It was anticipated from trial tests that lower replacements may not show the effects on the concrete–reinforcement bonding so it was decided to use only the higher replacements (100%) in order to find how severe this might affect the bonding. Mixing procedure complied with The Department of Environment, Design of Concrete Mixes [63]. The PBB test specimens were compacted on vibrating tables. Curing of the specimens was in accordance with BS 1881-111: 1983 [88].

The PBB specimens were covered with plastic sheets and left at room temperature for 24 hours before being stripped off their moulds and then left covered by wet cloth and plastic sheets at room temperature until the day of testing. 3 PBB test specimens for each mix were made with a total of 12 PBB tests.

Table 7-3: Compressive and tensile splitting strength for PBB specimens

	Specimen	f_{cu} (N/mm^2)	f_{ts} (N/mm^2)
Standard Concrete	B1	56.5	3.80
	B2	56.5	3.70
	B3	55.5	3.90
	Average	**56.0**	**3.80**
100% Recycled Concrete	B1	33.0	2.75
	B2	34.0	2.85
	B3	32.0	2.80
	Average	**33.0**	**2.80**
Standard Mortar	B1	45	3.90
	B2	46.5	3.95
	B3	45.5	3.80
	Average	**46.0**	**3.88**
100% Recycled Mortar	B1	32.0	2.40
	B2	33.5	2.80
	B3	30.5	2.65
	Average	**32.0**	**2.62**

7.2.2 Test Programme

Figure 7-4 shows the geometry and instrumentation for a typical PBB test specimen. The concrete compressive strength was obtained from 100 mm cubes as shown in Table 7-3. Tests to determine the tensile splitting strength were carried out on 150 mm diameter by 300 mm long cylinders, see Table 7-3. Testing of the PBB specimens was carried out at an age of 28 days.

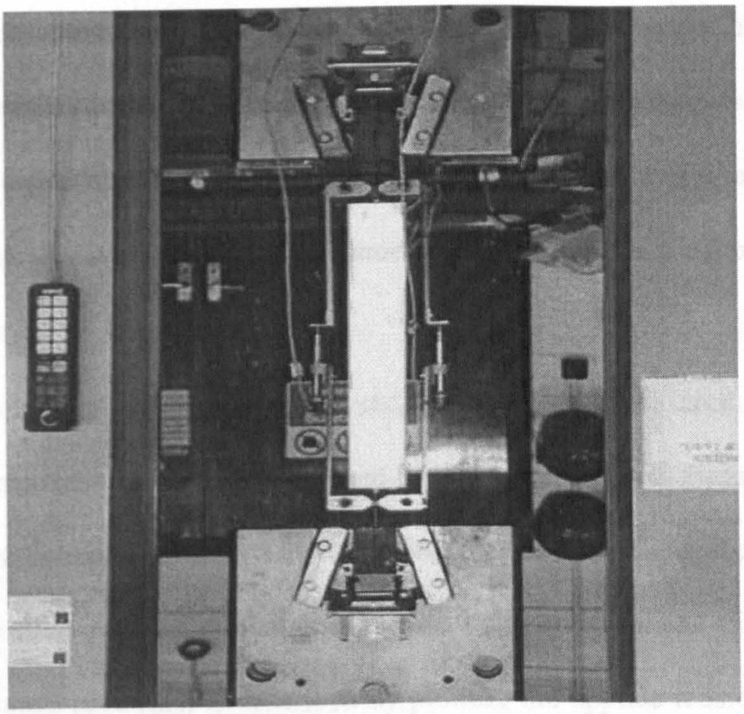


Figure 7-4: Prism bar bonding test using universal testing machine

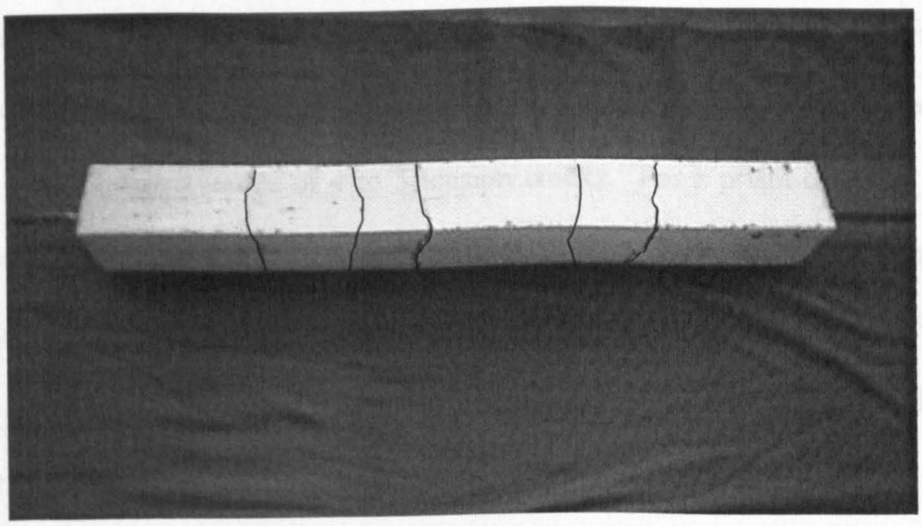


Figure 7-5: Typical tension cracks of a reinforcing bar bond test

The specimen was mounted vertically in a Zwick Universal testing machine that transmits the load through a set of tension grips on the protruding ends at the top and the bottom of the reinforcing bar. The rate of loading by the machine was 0.055 N/mm^2 per second (BS 1881 recommends a loading rate for flexural tension equivalent to 0.04 – 0.06 N/mm^2 per s.)

This resulted in tension being transferred from the steel reinforcing bar to the reinforced concrete element and caused tension cracks, see Figure 7-5. Two LVDTs were clamped to the steel reinforcing bar just outside of the concrete to measure the total elongation of the reinforced concrete specimen. The complete response of each specimen is described by plotting the applied tension versus the average member elongation by marking the cracks at its load stage, see Figure 7-6.

7.2.3 PBB Test Results of 10 mm Rebar

Figure 7-6 shows the tension cracks patterns developed at each load for a prism of standard concrete (see the Appendix 3 for all the specimens). It was expected that there would be an average of 4 to 5 tension cracks. For a prism of 100% RCA concrete a similar number of cracks occurred, however, it was noticed that the cracks initiated at a higher load compared to that of a standard prism, see Figure 7-7. For RCA prisms, the first tension crack appeared at a rebar stress value of about 250 N/mm^2 (±10 N/mm^2) that is higher than that of the standard prism of 190 N/mm^2 (±10 N/mm^2). This trend behaviour remained similar at higher stress values. It was estimated that the tension cracks for RCA prisms developed within a range of 50 to 70 N/mm^2 higher stress values than that of a standard prism.

This implies that the RCA have some effects and caused inferior bonding with the reinforcement. It is clear that, from Table 7-3, the tensile strength for the standard concrete is higher than that of 100% RCA concrete. Nevertheless, the force in the rebar when cast in standard concrete reaches its ultimate tensile strength at lower loads values compared to bar in the RCA concrete. Those cracks formations pointed towards a poorer bonding between the reinforcement and the RCA concrete. While in the standard concrete a better bonding facilitates the stress transformation to concrete causing tensioned cracked to develop at a lower load.

However, this behaviour was not reported for the mortar prism bars where both standard mortar and 100% RCFA developed the tension cracks at a different range of stress values, see Figure 7-8 and Figure 7-9. There was no clear trend behaviour in general but in some specimens the values tend to be similar or with slight differences. See all the specimens in Appendix 3

An experimental study carried out by Gustavson [89] on the bond response of wire strands and some influencing parameters found that the influence of the concrete strength on the bond capacity of the strand was hard to interpret. He explained that the density of the concrete matrix was found to be better parameter for determining the influence of the concrete rather than the strength. He found that an increased compressive strength of the concrete would not necessarily lead to an improved bond capacity.

To examine the results further, the ratio of the reinforcement bar force (P) against the concrete prism-beam tension force (Ft) were calculated, see Table 7-4, the lower the ratio is the better the bonding will be. The results showed that the bonding between the bars and the RCA concrete was not as good as the standard concrete. This was also confirmed by the RCFA mortar results, where the ratio (P/Ft) for mortar (0.99) is higher than the standard mortar (0.77); this means using RCFA causes poorer bonding with the reinforcement bars. Table 7-4 also shows the average bond stress (f_b N/mm^2) calculated at first crack for all the specimens; see Appendix 3 for all the data.

Table 7-4: Force on the reinforcement Bar (P) vs. Tension force at the concrete prism-beam (Ft)

AT FIRST CRACK	Average of all samples						
	Bar Force P (kN)	Stress (N/mm^2)	Development Length (mm) (for 1st crack)	f_{ts} (N/mm^2)	Beam tension Force Ft (kN)	P/Ft	f_b (N/mm^2)
Standard Concrete	15.21	193.63	171.53	3.80	21.38	0.71	2.89
100% RCA Concrete	19.28	245.43	177.13	2.80	15.75	1.23	3.48
Standard Mortar	16.85	214.43	195.27	3.88	21.84	0.77	2.82
100% RCFA Mortar	14.55	185.13	210.20	2.62	14.72	0.99	2.22

In another approach, Mitchell [90] explained that cracked concrete has the ability to decrease the strain in reinforcement due to tensile stress in concrete between the cracks. After cracking there is no tensile stress in the concrete at crack locations but there are tensile stresses in the concrete between the cracks. At the formation of the first cracks, the average tensile strength in concrete between the cracks will be reduced, and, as further cracks develop, the average stress in the concrete will be further reduced. Bond behaviour is a key aspect of the above as it controls the ability of the reinforcements to transfer tensile stresses to the concrete. From this concept, the tension versus elongation for standard concrete and that with 100% RCA were plotted as in Figure 7-10.

The area under the curves (the strain energy) for the prisms was calculated up to the line shown on the curves and then compared to the curve area of the reinforcement alone. Higher values of energy intake means better bonding.

Table 7-5 gives the ratios of the prism area curve to that of the reinforcement. These ratio values reflect the energy intake (credited to bonding) by concrete alone, similar things for mortar. Standard concrete prisms ratio (1.45) are slightly lower (about 5%) than that of 100% RCA prisms (1.52). However the mortar prisms reported to have larger differences in favour of the standard mortar where the strain energy ratio (1.54) is about 17% higher than that of 100% RCFA mortar (1.28). Clearly the ratio of the strain energy showed that the RCFA have more diverse effects on the bonding comparing to that with standard mortar.

Table 7-5: Curve area (strain energy) for 10 mm bar, concrete and mortars

	Test	Curve Area	Average	(A / As)
Reinforcement Bar 10 mm	T1	13.61	13.61	1.00
Standard concrete	T1	19.63	19.75	1.45
	T2	19.86		
100 % RCA concrete	T1	21.24	20.74	1.52
	T2	20.24		
Standard Mortar	T1	20.80	20.91	1.54
	T2	22.13		
100 % RCFA Mortar	T1	18.46	18.04	1.28
	T2	17.62		

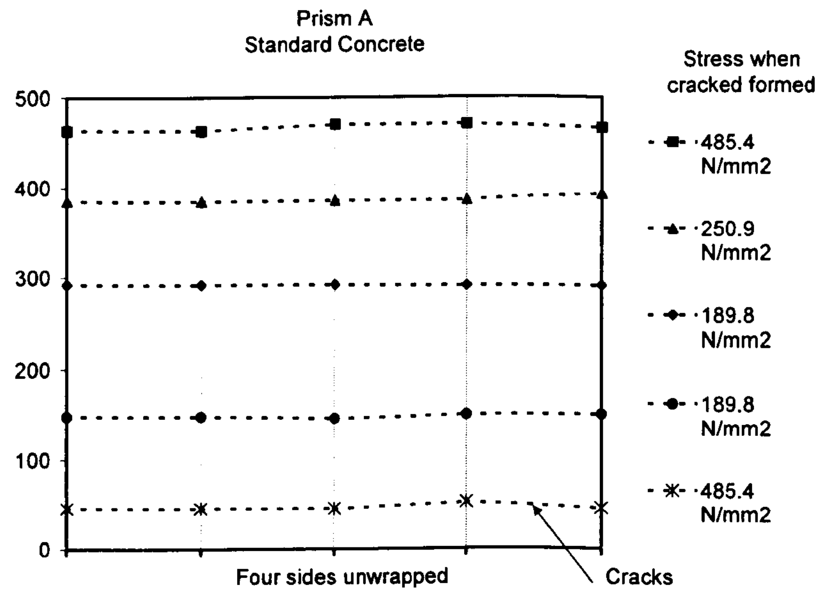


Figure 7-6: Tension cracks for standard concrete and 10 mm bar: Prism A

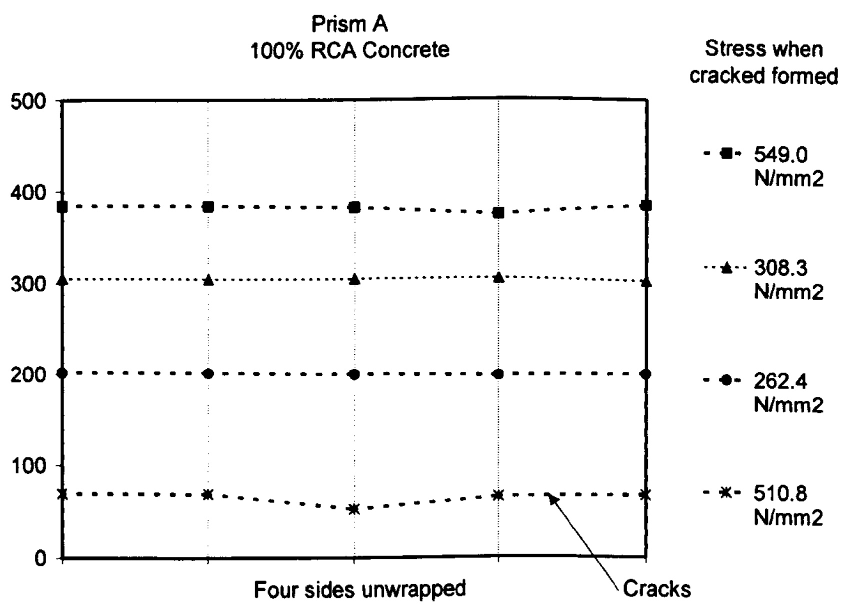


Figure 7-7: Tension cracks for RCA concrete and 10 mm bar: Prism A

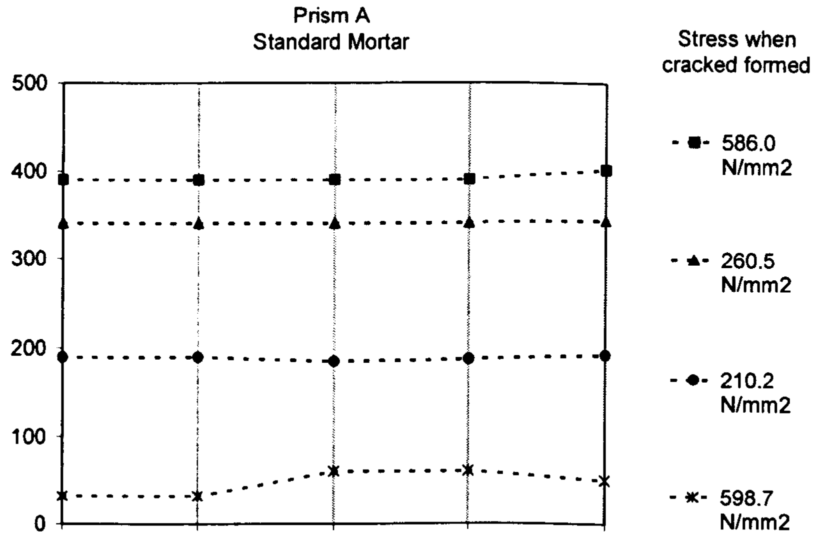


Figure 7-8: Tension cracks for standard mortar and 10 mm bar: Prism A

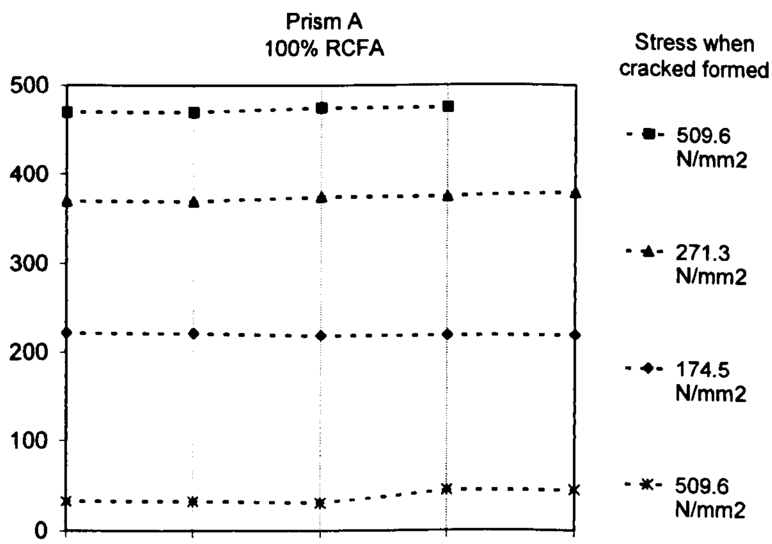


Figure 7-9: Tension cracks for RCFA mortar and 10 mm bar: Prism A

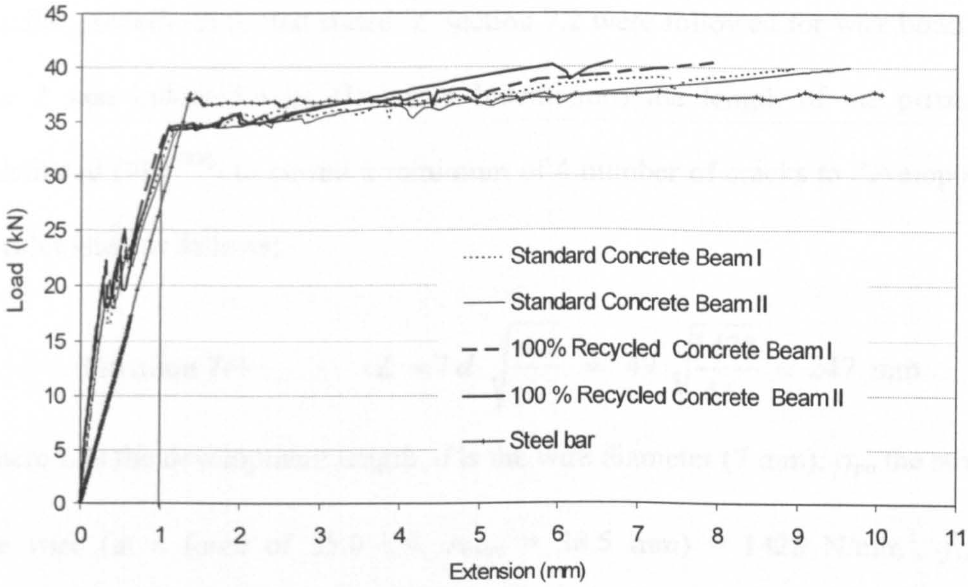


Figure 7-10: Tension versus extension responses of standard concrete and 100% recycled concrete with 10 mm bar

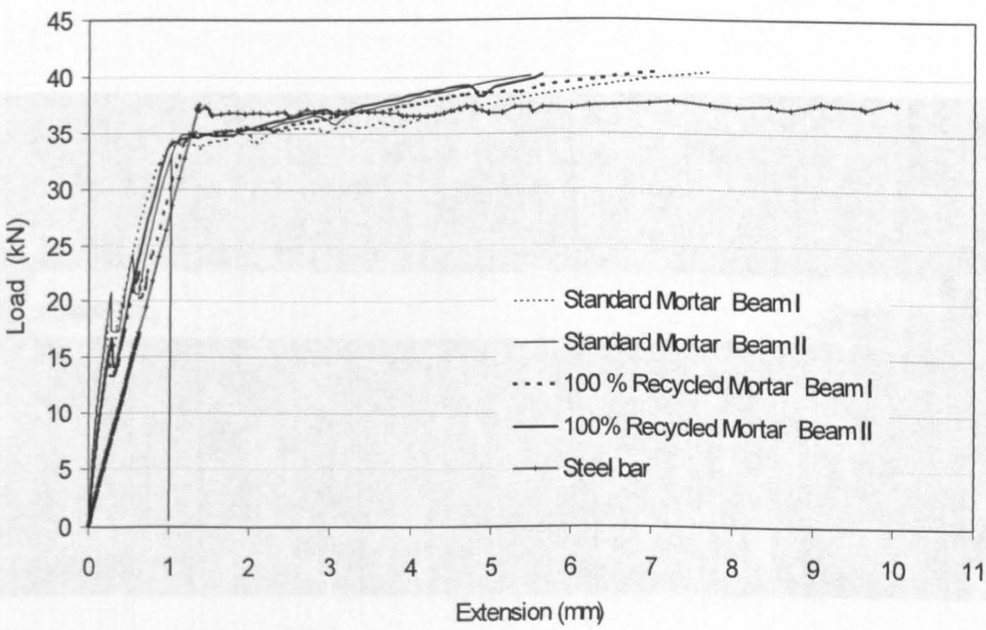


Figure 7-11: Tension versus extension responses of standard mortar and 100% recycled mortar with 10 mm bar

7.3 Pretensioning Wire Bond Tests

Similar procedures to that stated in Section 7.2 were followed for wire bond tests. For 7 mm indented wire (Belgium Indentation) the length of the prism was calculated (FIP [83]) to permit a minimum of 4 number of cracks to develop and it is calculated as follows:

Equation 7-4 $$L = 7\,d\sqrt{\frac{\sigma_{po}}{f_{cu}}} = 49\sqrt{\frac{1428}{56}} \approx 247 \text{ mm}$$

where L is the development length, d is the wire diameter (7 mm), σ_{po} the stress at the wire (at a force of 55.0 kN, A_{wire} = 38.5 mm) = 1428 N/mm^2, f_{cu} the compressive strength (56 N/mm^2 see Table 7-3). To ensure that at least two cracks are formed at each end of the specimen, the total length of the prism must be 5 times the development length = 5 x 247 = 1235 mm; Thus, a length of 1500 mm is adequate, see Figure 7-12.

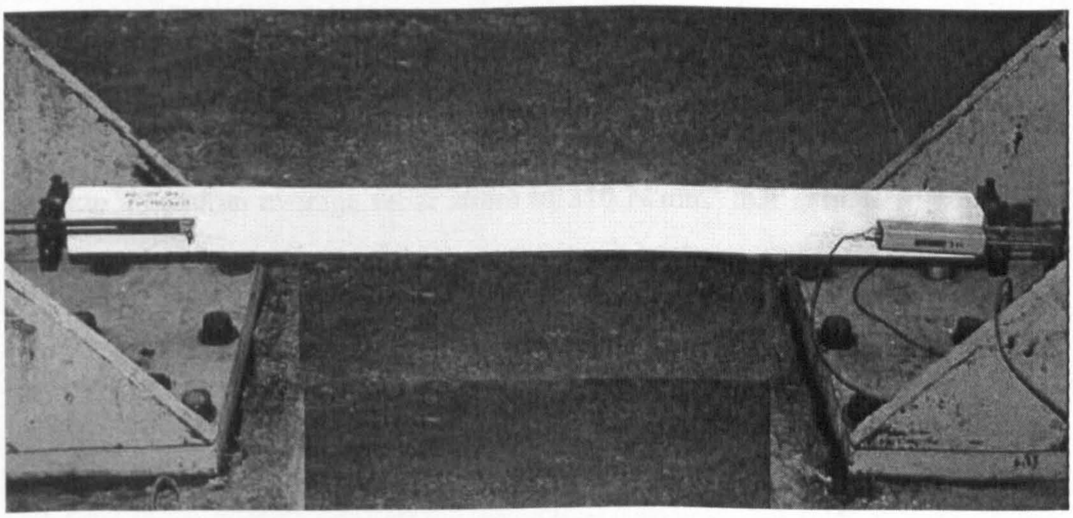


Figure 7-12: Loading frame for a prism wire bonding test

Mixing procedures and test programme were identical to that of the reinforcing bar bond tests see Sections 7.2.1 and 7.2.2. A different test setup was used because of the longer dimension of the wire bond test. The test setup consisted of a loading frame that transmits the load through a set of tension grips on the protruding ends at the top and the bottom of the reinforcing bar, see Figure 7-12. This resulted in tension being transferred from the steel prestressing bar to the reinforced concrete element. Two LVDTs were clamped to the wire just outside of the concrete to measure the total elongation of the reinforced concrete specimen. Similarly to the bar bond test, the complete response of each specimen is described by plotting the applied tension versus the average member elongation by marking the cracks at its load stage. 2 test specimens for each mix were made with a total of 8 prism wire bonding tests.

7.3.1 PBB Test Results of 7 mm Pretensioning Wire

Figure 7-13 shows an average of 4 to 5 cracks developed in a wire prism of standard concrete (see Appendix 3 for all the specimens). The cracks started to develop around an average rebar stress of 310 N/mm^2 that is higher than that of 100% RCA concrete (around 244 N/mm^2), see Figure 7-13 and Figure 7-14. The reinforcing bar tests (Section 7.2.3), the RCA concrete with wire reached the tensile capacity at lower loads and before that of the standard concrete. This could imply that the RCA concrete bonded well with the wires and helped to transfer the strain to the RCA concrete where upon the tensile cracks developed. However it should be pointed out that the tensile strength of the RCA concrete (100%) is lower than that of standard concrete (Table 7-3) and this could cause it to cracks at earlier loads compared to the standard concrete.

Similar findings were reported for the mortar prism. Figure 7-15 shows tensile cracks for the standard mortar prism starting to develop around a wire stress of 636 N/mm^2. This is also higher than that of RCFA mortar (around 363 N/mm^2), see Figure 7-16. (See Appendix 3 for all the specimens).

The ratio of the prestress wire force (P) against the concrete prism-beam tension force (Ft) were calculated, see Table 7-6. Similar to the reinforcement bar, standard concrete showed lower ratio (better bonding) than RCA concrete, however standard mortar reported showed the opposite 13% higher ratio poorer bonding than RCFA mortar. Table 7-6 also shows the average bond stress (f_b N/mm^2) calculated at first crack for all the specimens, see Appendix 3 for all the data.

Table 7-6: Force on the prestress wire (P) vs. Tension force at the prism-beam (Ft)

AT FIRST CRACK	Average of all samples						
	Bar Force P (kN)	Stress (N/mm^2)	Development Length (mm) (for 1st crack)	f_{ts} (N/mm^2)	Beam tension Force Ft (kN)	P/Ft	f_b (N/mm^2)
Standard Concrete	10.6	275.5	452.0	3.8	21.1	0.5	1.2
100% RCA Concrete	8.26	214.65	543.50	2.80	15.75	0.53	0.73
Standard Mortar	20.1	522.1	692.5	3.9	22.1	0.9	1.6
100% RCFA Mortar	11.50	298.80	476.70	2.60	14.63	0.78	1.10

Considering the tension versus elongation (the strain energy), the standard concrete showed higher strain energy ratio (=1.17) than that of the RCA concrete (=1.09), see Figure 7-17 and Table 7-7. Similarly for mortars the strain energy ratio for standard mortars was 1.32 and that of 100% RCFA was 1.07, see Figure 7-18 and Table 7-7. Although the crack patterns did not state the effects clearly, the strain energy values indicate that the recycled aggregate has evidently affected the bonding with the wire.

Table 7-7: Curve area (strain energy) for 7mm wire, concrete and mortars

	Test	Area	Average	(A / As)
Pretensioning 7 mm Wire	T1	253.77	250.37	1.00
	T2	246.98		
Standard concrete	T1	289.18	291.81	1.17
	T2	294.45		
100 % RCA concrete	T1	266.04	271.7	1.09
	T2	277.36		
Standard mortar	T1	312.96	331.27	1.32
	T2	345.58		
100 % RCFA mortar	T1	268.58	268.58	1.07
	T2	-		

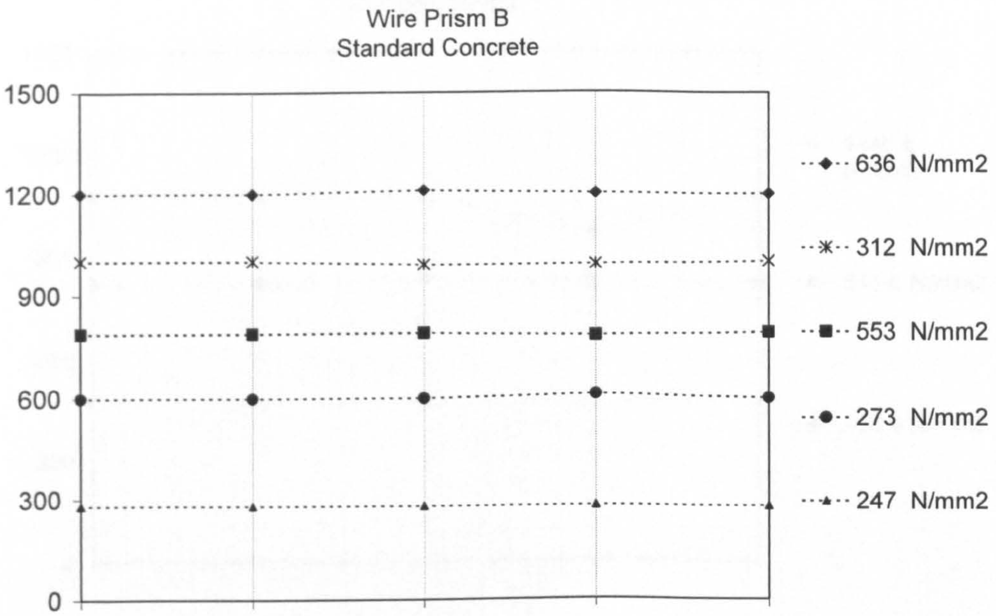


Figure 7-13: Tension cracks for standard concrete 7mm wire Prism A

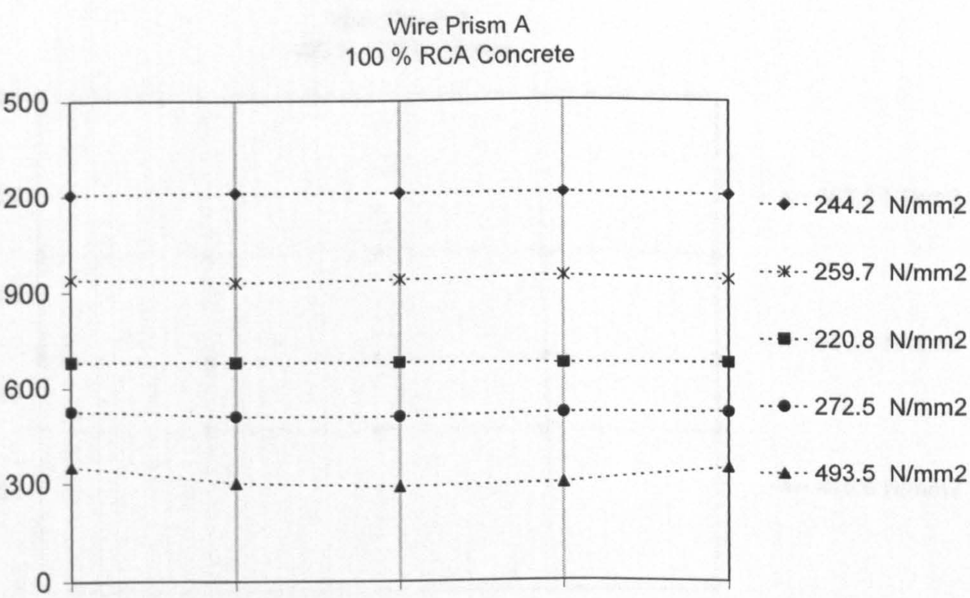


Figure 7-14: Tension cracks for RCA concrete and 7mm wire: Prism A

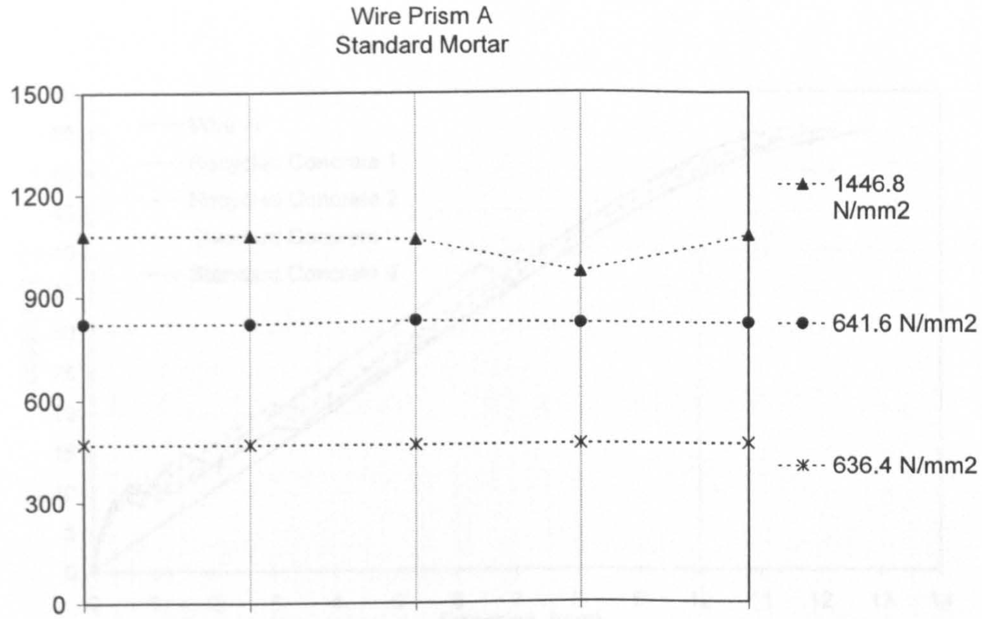


Figure 7-15: Tension cracks for standard mortar and 7mm wire: Prism A

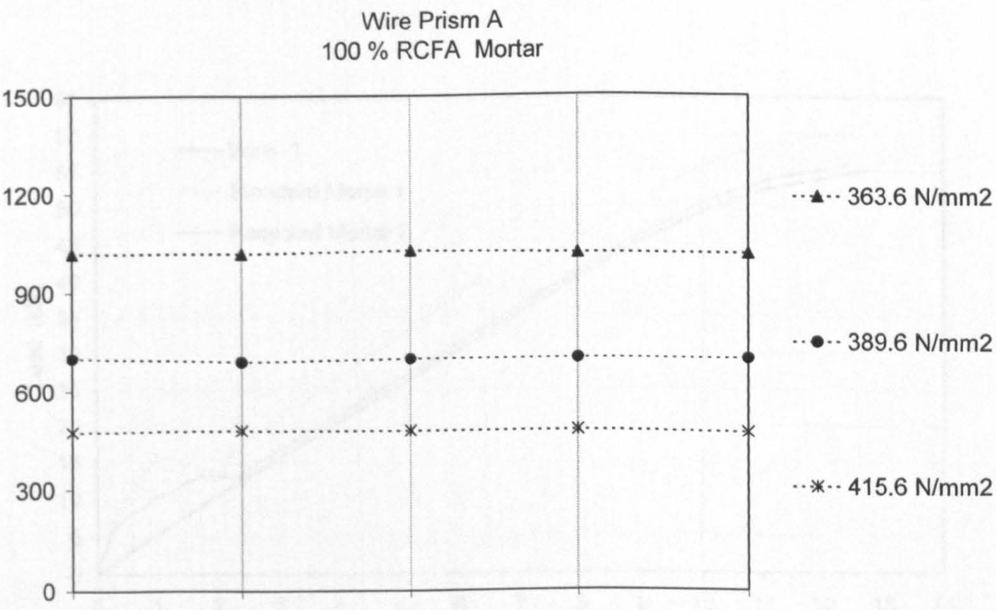


Figure 7-16: Tension cracks for RCFA mortar and 7mm wire: Prism A

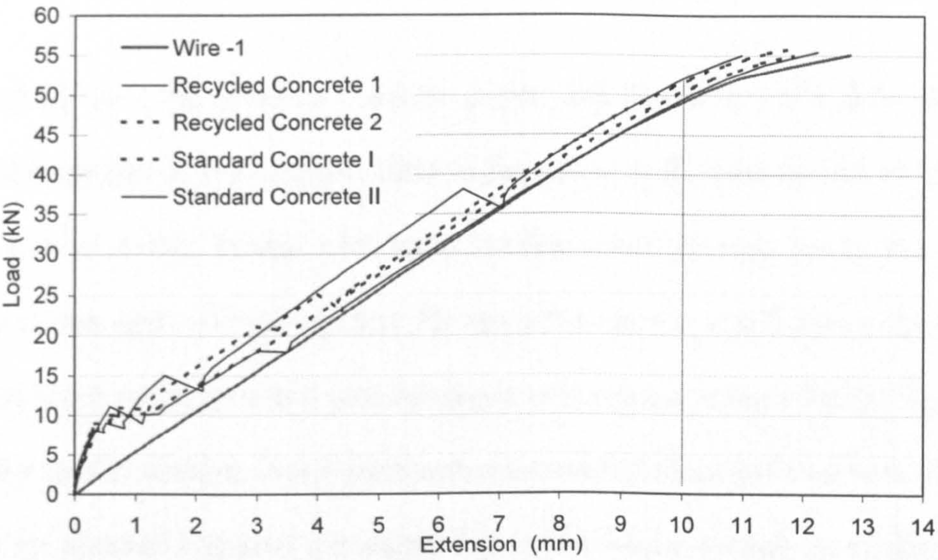


Figure 7-17: Tension versus extension responses of standard concrete and 100% recycled concrete with 7-mm wire

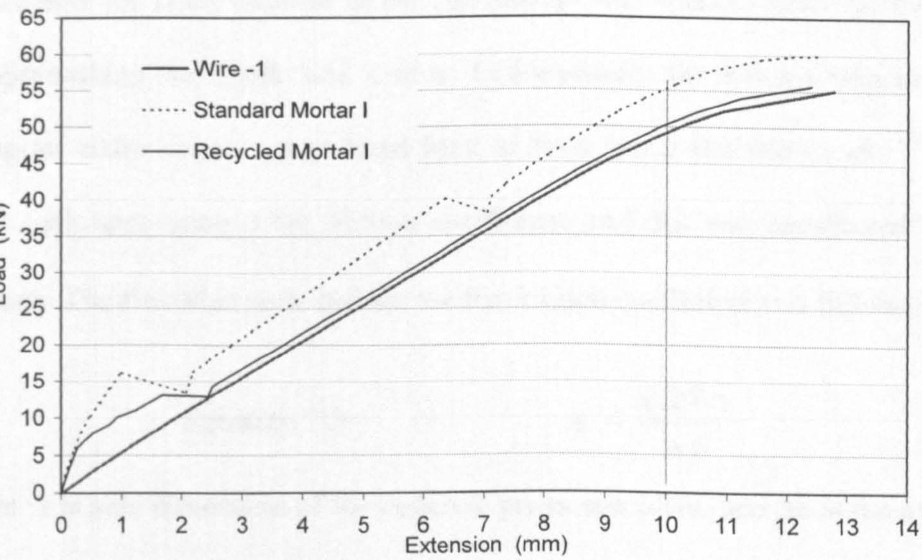


Figure 7-18: Tension versus extension responses of standard mortar and 100% recycled mortar with 7-mm wire

7.4 Discussion

Although both the standard concrete prisms and that with 100% RCA showed similar number of cracks, they differ in the values of the load needed to develop the tension cracks. Prisms with better bonding could develop cracks at a lower load to that with a lower bonding. However the same is true for concrete with a lower tensile strength which will develop tensile crack at a lower load to that with higher tensile strength. For prisms with reinforcement bars crack patterns showed that the standard concrete, although it has higher tensile strength, performs better in bonding and developed cracks earlier than that of RCA concrete.

However, prisms with wires showed that RCA concrete developed cracks earlier than standard concrete. This could be related to a lower tensile strength of RCA concrete and possibly a good bonding with wires.

To consider the crack patterns further, an attempt was made to interpret the result by approaching Standards and Codes. Unfortunately the British Standard and European codes do not adopt these kind of tests, but a Brazilian code [91] does adapt such tests using a bar friction coefficient, and this was considered in this research. The Brazilian code defines the bar friction coefficient η as follows:

Equation 7-5

$$\eta = \frac{2.25\ s}{\Delta c}$$

where s is side dimension of the concrete prism specimen, and Δc is the average distance between cracks measured on the four faces of the specimen. A higher bar friction coefficient means greater bond strength.

Table 7-3 showed that the tensile splitting strength for concrete with a 100% RCA reduced by about 26%. From this reduction at higher replacements one could anticipate that the bond strength for concrete with RCA would be considerably lower than that of the standard concrete.

However, the following test results showed only a slight reduction occurred. Table 7-8 shows the friction coefficient average values for standard concrete mixes and that with 100% RCA. See Appendix 3 for the full data. It was found that 100% RCA slightly reduced the bond strength in terms of the bar friction coefficient, which for recycled concrete is 2.71, about 10% lower than that of standard concrete of 3.02. For mortar mixes, although lower values for bar friction coefficients were reported than for coarser aggregates, see Table 7-9. Apart from one RCFA specimen B3 that had a surprising high value $\eta = 3.88$, but there was no significant difference in η between natural mortar (2.40) and 100% RCFA (2.53).

Table 7-10 shows the friction coefficient average values for pretensioning wires in standard concrete and that with 100% recycled aggregate. (See Appendix 3 for the full data). Standard concrete showed slightly lower friction coefficient value (0.87) than that of the RCA concrete (0.97). This implies that wires bonded well with RCA concrete and this agrees with the cracks patterns findings, but contradicts the strain energy findings which showed that standard concrete performed better in bonding. The coefficients for the standard and recycled mortars are much closer, 0.97 and 0.91, respectively. The slightly improved bonding of the standard mortar agrees with the strain energy findings.

Table 7-8: Reinforcement bar friction coefficient values for concrete mixes

	Specimen	Δ (c)	η = 2.25 (s /Δ c)
Standard Concrete	B1	84	2.68
	B2	82	2.74
	B3	62	3.63
	Δ B	**76**	**3.02**
100% Recycled Concrete	B1	80	2.81
	B2	70	3.21
	B3	107	2.10
	Δ B	**86**	**2.71**

Table 7-9: Reinforcement bar friction coefficient values for mortar mixes

	Specimen	Δ (c)	η = 2.25 (s /Δ c)
Standard Mortar	B1	86	2.62
	B2	101	2.23
	B3	95	2.37
	average	**94**	**2.40**
100% Recycled Mortar	B1	110	2.05
	B2	136	1.65
	B3	58	3.88
	average	**101**	**2.53**

Table 7-10: Wire friction coefficient values for concrete mixes

	Specimen	Δ (c)	η = 2.25 (s /Δ c)
Control Concrete	B1	210	0.80
	B2	180	0.94
	average	**195**	**0.87**
100% Recycled Concrete	B1	180	0.94
	B2	170	0.99
	average	**175**	**0.97**

Table 7-11: Wire friction coefficient values for mortar mixes

	Specimen	Δ (c)	η = 2.25 (s /Δ c)
Control Mortar	B1	160	1.05
	B2	190	0.89
	average	**175**	**0.97**
100% Recycled Mortar	B1	190	0.89
	B2	180	0.94
	average	**185**	**0.91**

In a related study Jianzhuang and Falkner [92] investigated the bond behaviour between recycled aggregate concrete and steel rebars and tried to establish a bond stress versus slip relationship between recycled aggregate concrete and steel rebars. Three different RCA replacement percentages were used in their study 0%, 50% and 100%, respectively. The recycled coarse aggregate was obtained by processing waste concrete from the runway of an airport in Shanghai. The natural coarse aggregate was common crushed stone. The water/cement ratio was kept constantly to 0.43 however the RCAs in their investigation were pre-soaked by additional water before mixing and the amount of this additional water was calculated on the basis of the saturated surface-dry condition. Using standard pull out test they have concluded that the bond strength between the recycled aggregate concrete and steel rebars is higher than the one between normal concrete and steel rebars and they stated that the anchorage length of steel rebars embedded in the recycled aggregate concrete with 100% RCA can be chosen as the same for normal concrete under the condition of the same compressive strength of concrete. They also found that the general shape of the load versus slip curve between recycled aggregate concrete and steel rebars is similar to the one for normal concrete and steel rebars. They explained their findings on the possibility that the values of modulus of elasticity of the recycled coarse aggregate and the cement paste of the recycled aggregate concrete might be similar but no further explanation were given.

7.5 Conclusions

Two methods were mainly used to interpret the data obtained from the prismatic bar bonding tests. The first was based on the crack patterns and the average distance between the cracks using the bar friction coefficient values. The second was using strain energy obtained by plotting the tension against the elongation.

For the reinforcing bars both methods indicated that 100% RCA concrete reduced the bonding. However only the strain energy ratio indicated a good behaviour for RCA concrete. In mortars most tests reported a similar behaviour for standard mortar and that with 100% RCFA.

For the pretensioning wires, although most methods indicated that 100% RCA bonded well with wires, the strain energy ratio showed a poorer performance in bonding than the standard concrete. (This poorer effect will be exposed again by the X-beam tests in Chapter 8).

In mortars both methods concluded that the RCFA affected the bonding and showed that the wires did not perform in bonding as well as in standard mortar.

It should be noted that although high proportions of recycled aggregates were used in prismatic bar and wire bonding tests (100% of RCA in concrete and 100% RCFA in mortars) the tests showed only slight reductions in bonding and in some cases it showed better performance. Bearing in mind the fact that most precast industries used only limited proportions of RCCA (20% to 30%) the bonding test result gives an early indication that using these proportions may not affect the bonding and may only cause accepted marginal reductions compared to standard concrete. The next chapter will test these findings further.

8 Flexural and Shear Behaviour of Prestressed X-Sections Beams with RCA

Introduction

The effects of using RCA on concrete performance, compression and flexure, had been studied in chapter 6 and then followed by an investigation on the bond between reinforcement and concrete in chapter 7. These investigations provided some indication of how the RCA could affect prestressed concrete. To fulfil this work, and since hollow core slab units was the parent material, then it is of particular interest to investigate these RCA in some experimentally convenient beams that have a cross section resembling part of a hollow core slab units. To accomplish this, a series of tests has been made on X-shaped sections, called 'X-beams', of concrete containing prestressing reinforcement. The X-shape was chosen because it closely simulates the rounded webs of an extruded hollow core slab units (see Figure 8-1). For X-beam design detail see Appendix 4. The aim was to assess the effects of adding proportions of RCA on the flexural performance of prestressed concrete. Three points loading was used as shown Figure 8-2. The strain losses and deflections were recorded and discussed. The strain on the concrete surface and in the wires were monitored and compared to standard concrete.

8.1 Mix Design

Two sets of test have been used for the X-beam flexural tests, see Table 8-1. In set one, concrete mixes were used where both the coarse and fine aggregates are replaced with 20% and 50% RCCA and RCFA. In set two, 20% and 50% RCCA were used to replace coarse aggregates only. RCCA and RCFA were obtained using jaw and impact crushers respectively. Although bond tests (Chapter 7) were carried out on 100% RCA in order to determine the maximum effect on bonding, lower percentages were used here to be consistent with the compression and tension tests in earlier mixes described in Chapter 5. The mixing procedure complied with The Department of Environment, Design of Concrete Mixes [63]. The X-beams were compacted using an external shutter vibrator and on some occasions, it was necessary to use a poker vibrator.

Table 8-1: Schedule of mixes and tests for X-beams

Type	Replacements	Reference no.	Compressive Strength	
			Detension	Testing
Natural	-	NA-1	36.0	53.0
		NA-2	38.0	54.0
RCA	20% RCA	20RCA1	38.0	53.0
		20RCA2	37.0	56.0
	50% RCA	50RCA1	38.0	50.0
		50RCA2	36.0	49.0
RCCA	20% RCCA	20RCCA1	37.0	56.0
		20RCCA2	40.0	57.0
	50% RCCA	50RCCA1	36.0	54.0
		50RCCA2	39.0	53.0

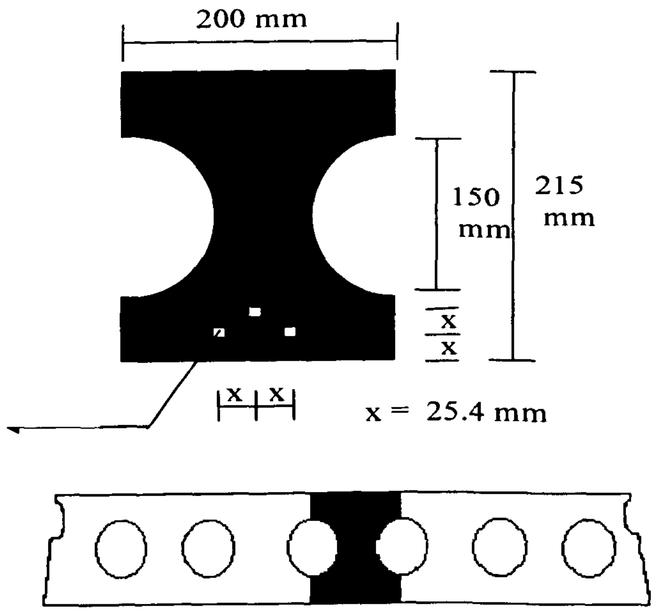


Figure 8-1: Typical hollow core slab units cross section and X-beam tests specimen.

Curing of the specimens was in accordance with BS 1881-111: 1983 [88]. The X-beam specimens were covered with polythene wet hessian sheets and left at room temperature up to de-tensioning then left uncovered in air at room temperature until the day of testing. For repetition and confirmation, two specimens of X-beam for each mix were made, making a total of 10 tests.

8.2 Prestressing

The laboratory prestressing bed was used to cast the X-beams. Prestress was applied using a Pilcon Super 7 stressing jack operated by a manual hydraulic pump. The configuration of the prestressing wires is shown in Figure 8-1. One type of prestressing wire of nominal 7 mm diameter, with "Belgian Indentations" and conforming to BS 5896:1981 [93] was used. Strain gauges were attached on the top wire and it was necessary to grind the indent smooth.

As protection against ingress of water and abrasion at release of the prestress the strain gauges were coated with two or more layers of epoxy resin. Three electrical resistance strain gauges were used to check the stress in the wires, and to measure the prestress losses. The gauges were placed at distances of 333, 666 and 1000 mm from one end of the beam see Figure 8-2. Each of the 7 mm wires (A_{ps} = 38.5 mm^2) was stressed to 45 kN (0.70 x ultimate tensile strength f_{pu} = 1670 N/mm^2), See Appendix 4 for the X-beam design. The prestressing force was released 3 days after casting (see Table 8-1 for the compressive strength values). The release of stress was by the slow method of gradually reducing the distance between the prestressing blocks.

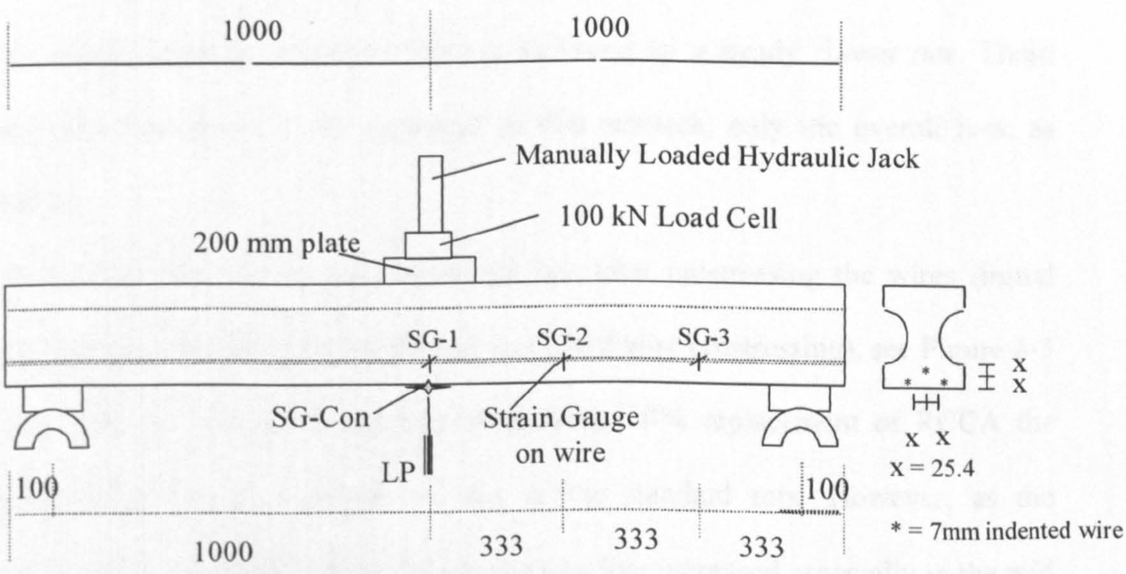


Figure 8-2: X-beams for three point flexural test

8.3 Prestressing loss

Prestressing loss in the wires was the initial investigations on the X-beams with RCA. Generally the losses of the prestressing force can be grouped into two categories, (i) those which occur immediately during construction of the beam and after transfer of pretension and (ii) those which occur over an extended period of time. The prestressing jack force is immediately reduced by losses due to relaxation of tendons, friction and anchorage slip. This is not affected by the RCA as it is mainly a mechanism loss. Upon release the beam undergoes elastic shortening due to the transfer of force from the wires into the concrete. This loss, typically about 3-5%, is a function of the Young's modulus of concrete at the time of release. The main concern was the loss between de-tensioning and the test day where the concrete takes control and the RCA is observed to have some effect due to shrinkage and the early stages of creep. Usually during the initial stages there is a rapid reduction in stress, which is followed by a steady slower rate. These individual losses were not measured in this research, only the overall loss, as follows.

The prestressing strains were measured just after prestressing the wires (initial wire prestressing strain) up to the test day (final wire prestressing), see Figure 8-3 and Table 8-2. It was found that when using 20% replacement of RCCA the prestressing loss was similar to that of the standard mix. However, as the replacement increased to 50% the prestressing loss increased especially in the mid span region (SG-1).

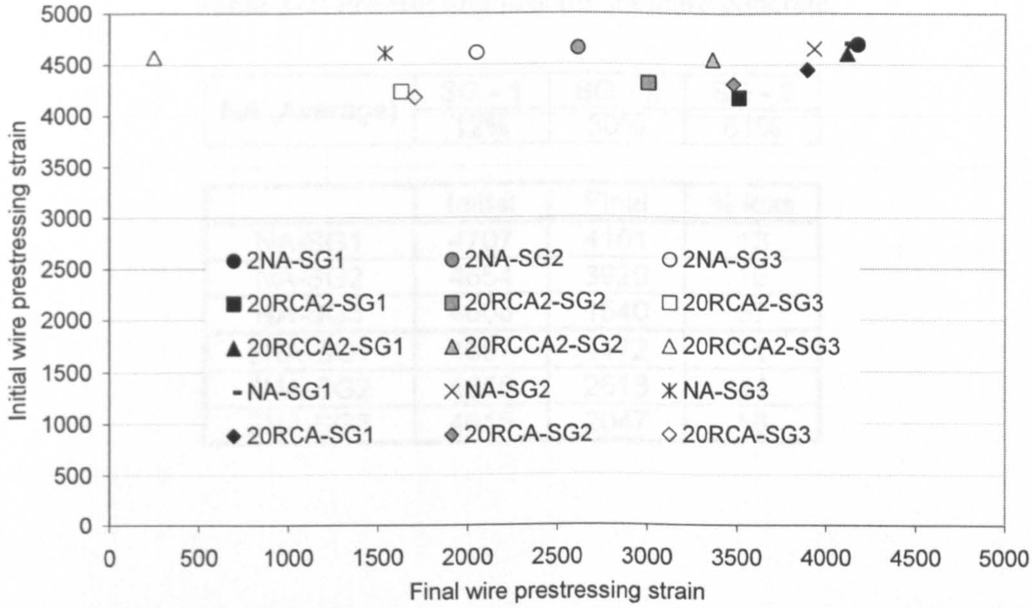


Figure 8-3: Prestressing loss in wires ($x10^{-6}$) with 20% replacements

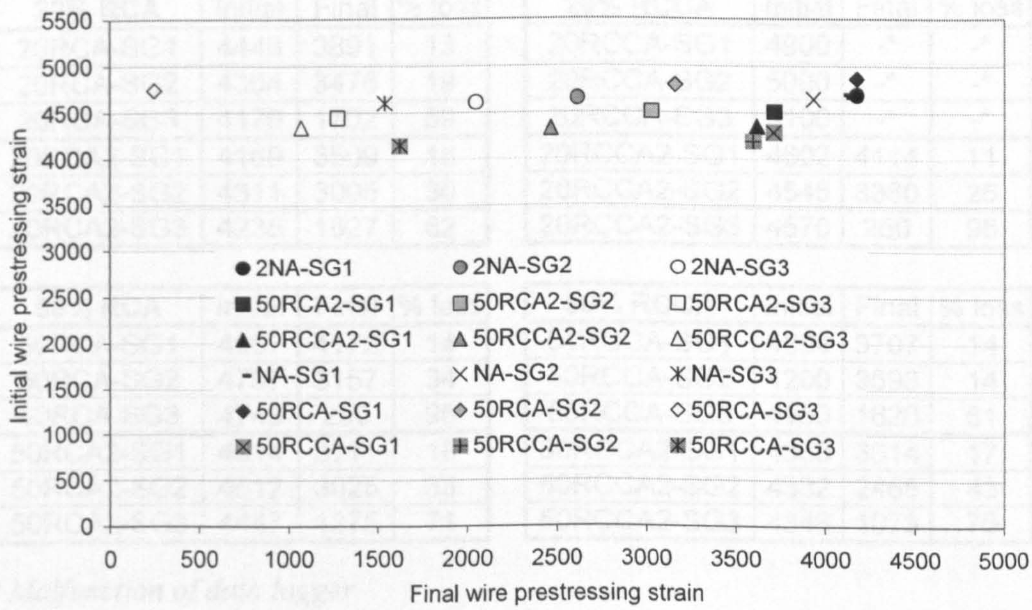


Figure 8-4: Prestressing loss in wires ($x10^{-6}$) with 50% replacements

Table 8-2: Prestressing loss for standard concrete

NA (Average)	SG - 1	SG - 2	SG - 3
	12%	30%	61%

	Initial	Final	% loss
NA-SG1	4707	4101	13
NA-SG2	4654	3929	16
NA-SG3	4608	1540	67
2NA-SG1	4697	4172	11
2NA-SG2	4656	2613	44
2NA-SG3	4615	2047	56

Table 8-3: Prestressing loss for concrete with RCCA and RCA.

Average	SG - 1	SG - 2	SG - 3
20% RCA	14	25	60
50% RCA	16	34	83

Average	SG - 1	SG - 2	SG - 3
20% RCCA	11	26	95
50% RCCA	15	29	68

20% RCA	Initial	Final	% loss
20RCA-SG1	4448	3891	13
20RCA-SG2	4304	3476	19
20RCA-SG3	4176	1702	59
20RCA2-SG1	4169	3509	16
20RCA2-SG2	4311	3005	30
20RCA2-SG3	4235	1627	62

20% RCCA	Initial	Final	% loss
20RCCA-SG1	4900	-*	-*
20RCCA-SG2	5000	-*	-*
20RCCA-SG3	5100	-*	-*
20RCCA2-SG1	4602	4114	11
20RCCA2-SG2	4545	3360	26
20RCCA2-SG3	4570	250	95

50% RCA	Initial	Final	% loss
50RCA-SG1	4867	4172	14
50RCA-SG2	4797	3157	34
50RCA-SG3	4745	250	95
50RCA2-SG1	4514	3711	18
50RCA2-SG2	4512	3025	33
50RCA2-SG3	4447	1275	71

50% RCCA	Initial	Final	% loss
50RCCA-SG1	4294	3707	14
50RCCA-SG2	4200	3593	14
50RCCA-SG3	4150	1620	61
50RCCA2-SG1	4358	3614	17
50RCCA2-SG2	4332	2465	43
50RCCA2-SG3	4346	1073	75

* *Malfunction of data logger*

NA means samples one of standard concrete and 2NA means samples two, similarly for RCA and 2RCA. SG1, SG2 and SG3 related to strain gauges as in Figure 8-2.

When combining RCFA and RCCA, the result clearly showed further loss in the highest prestressing zone compared to the standard and RCCA mixes, see Figure 8-4 and Table 8-3. The prestressing loss also increased as the replacement increases. This agrees with the findings from the bond tests (Section 7.4), and will also be noticed further in the X-beams flexural failure, which will be discussed in later sections.

8.4 Testing

The beams were simply supported over effective spans of 1800 mm whilst a bearing of 100 mm was used at both ends of the beam to imitate typical hollow core slab units applications. The set-up is shown in Figure 8-5. A linear-potentiometer (LP) was placed under the beam to measure deflections at the mid-point (load-point). A concrete strain gauge was attached to the concrete bottom surface placed at the mid-point to measure the concrete strains. Load was applied by a hydraulic jack attached to a manual pump and controlled with a 100 kN load cell.

Fibreboard was used between the load plate and beam to compensate for the uneven surface and provide a level-loading platform. The load was applied in increments of 3 kN up to 15 kN and then with increments of 1 kN used up to the end. A data logger connected to a personal computer automatically recorded the load, deflections and strains. Live plots of load versus deflection were monitored throughout the tests. During each of the tests, wire slip was measured using a depth gauge. Approximately 15 days after casting the X-beams were tested; see Table 8-1 for the compressive strength.

8.5 Flexural Failure Values (or Cracking & Ultimate Loads Values)

Table 8-4 shows the values at which both flexural and shear failure happened while loading the X-beams. General behaviour for all the specimens starts by a single flexural crack occurring approximately under the loading point, propagating rapidly vertically through the narrow web and causing the X-beam to enter a post-cracking stage where the second moment of area decreases rapidly, thus increasing the strain in the wires. The flexural cracking moment was calculated as follows:

$$\text{Equation 8-1:}\quad M_{cr} = [(f_{bc} + 0.45\sqrt{f_{cu}}) \times Z_b] = (f_{bc} + f_{ct}) \times Z_b$$

f_{bc} the final stress after loss and it was calculated using the actual loss, f_{ct} the average flexural tensile strength, Z_b elastic section modulus at bottom fibres. The beams were designed to crack flexurally at 17.8 kNm. However Equation. 8-1 was also used to calculate M_{cr} where f_{bc} is determined from actual losses, resulting in values of M_{cr} = 19.55 - 20.43 kNm. See Table 8-4 and Appendix 4 for full design details.

As the loading continued typical flexural crack occurred around the middle span and propagated up towards the loading point see Figure 8-6. At higher loads a sudden flexural shear crack occurred that propagated at 45° to the horizontal toward the edge of the bearing and the loading point, see Figure 8-7. This sudden flexural shear failure occurred because of the following factors: (a) the stress in the concrete between the support and the loading point increased significantly, especially after the flexural crack had propagated, (b) the absence of shear reinforcement, and (c) the lack of dowel action due to a small diameter of wires.

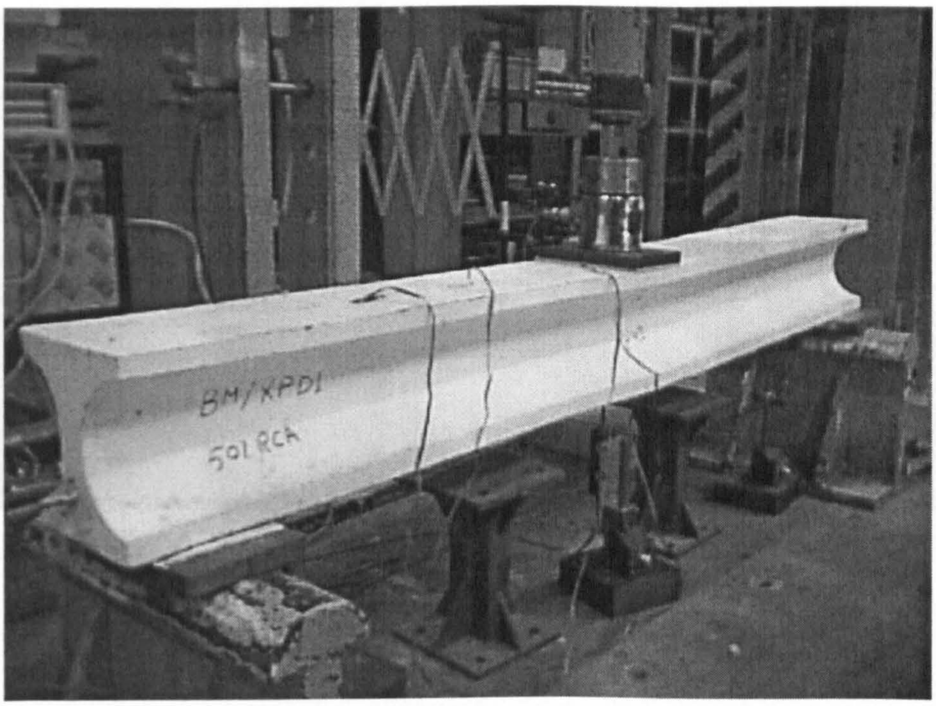


Figure 8-5: Typical photo of X-beams for three point flexural test

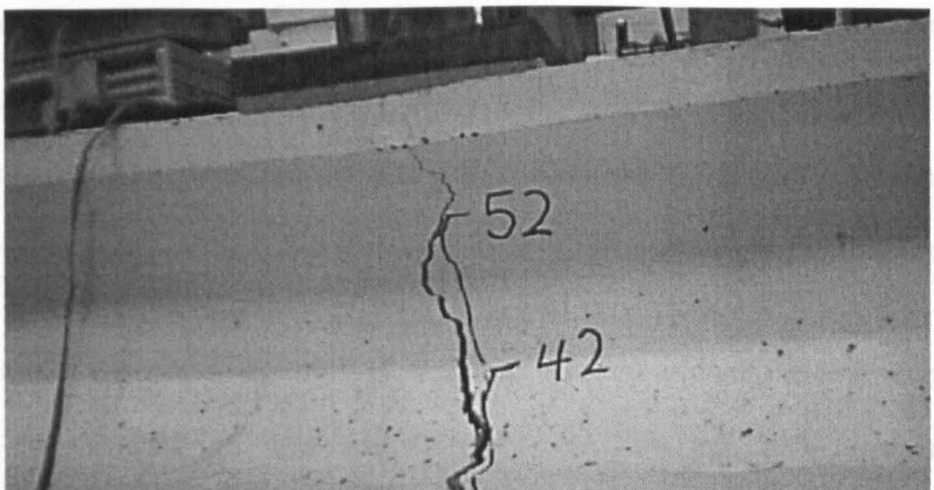


Figure 8-6: Typical flexural cracks for prestressed X-beams

Figure 8-7: Typical flexural shear crack failure for prestressed X-beams

The values at which the cracks occurred varied based on the replacement percentages of the RCCA or RCA (Table 8-4). For standard X-beam the crack occurred initially due to flexure at an average bending moment of 20.3 kNm and propagated into the compression flange until a sudden critical flexural shear cracks occurred at a bending moment of 28.5 kNm. After this shear crack the load carrying capacity of the X-beam reduced sharply indicating a complete failure.

Adding RCCA causes the flexural cracks to occur at a lower bending moments than the standard, i.e. 19.4 and 18.9 kNm at 20% and 50% replacement, respectively. Correspondingly, the RCCA X-beams failed at moments of 23.6 and 22.0 kNm, some 20-30% lower than the standard X-beam (28.5 kNm.). Combining RCCA and RCFA caused the X-beams to propagate flexural cracks at lower bending moments (18.5 and 18.0 kNm at 20% & 50% RCA) than the values of both the standard and RCCA X-beams. However, complete shear failure was in a similar range to the RCCA X-beam, and 22% lower than the standard X-beam.

Clearly the standard X-beams performed well and cracked at a higher value than the design value though the ultimate failure was slightly lower than the calculated one due to shear failure not allowing the calculated ultimate moment (M_{ur} = 31.1 kNm) to develop fully. Any reduction in flexural strength will cause a reduction in cracking resistance of the beam. Furthermore, although the bond tests were carried out at higher replacements, they indicate, by virtue of the strain energy ratio, that RCFA depreciates the bonding with the wires. For example, the final strain near to the end of the RCA beams (Figure 8-3 & 8-4; SG-3) was less than 1600 $\mu\varepsilon$, i.e. a residual wire stress of less than 328 N/mm^2 (based on E_s = 205 GPa), compared with more than 800 N/mm^2 in the standard beam at mid-span.

The effect this has on f_{bc} would have contributed to the lower flexural-shear failure strength in the RCA beams. The fact that the concrete strengths at transfer and testing (Table 8-1) were not more than ±4 N/mm^2 to the standard beams leads to the conclusion that the failure load in a prestressed X-beam is dominated more by the effect recycled aggregates have on final prestress than the structural behaviour *per se*. This agrees with Gustavson [89] who found that that the compressive strength of the concrete is not a relevant parameter for describing the concretes influence to the bond capacity of strands. He explained that the density of the different concrete matrixes is a better parameter to use and the bond capacity is more dependent upon the friction and adhesion between the rod's surface and the surrounding concrete. Previous tests in chapter 6 proved that RCA concrete has lower densities than the standard concrete, which caused the concrete to have less friction and adhesion to the wire strands.

Table 8-4: Actual and calculated loads & moments at initial cracks and at ultimate failure for X-beams

	Design Moment kNm	Flexural Cracks kNm		Ultimate Failure kNm	
		Calc Mom with actual loss M_{cr}	Test Moment	*Calculated Ultimate* M_{ur}	Test Ult Failure
NA	17.8	20.28	20.3	31.1	28.5
20% RCA		19.55	18.5		23.1
50% RCA		20.10	18.0		23.5
20% RCCA		20.43	19.4		23.6
50% RCCA		19.92	18.9		22.0

X-beam	Flexural Cracks		Cracks Flexural Shear Type	
	Load (kN)	Moment (kNm) *Average*	Load (kN)	Moment (kNm) *Average*
NA	46	**20.3**	63	**28.5**
NA2	44		64	
NA (average)	**45**		**63**	
20RCA	40	**18.5**	52	**23.1**
20RCA2	42		51	
20RCA (average)	**41**		**51**	
50 RCA	40	**18.0**	51	**23.5**
50RCA2	40		54	
50RCA (average)	**40**		**52**	
20RCCA	42	**19.4**	54	**23.6**
20RCCA2	44		51	
20RCCA (average)	**43**		**52**	
50RCCA	40	**18.9**	49	**22.0**
50RCCA2	43		49	
50RCCA (average)	**42**		**49**	

8.6 Deflection and Strain on Concrete and Prestressing Wires

8.6.1 Deflection

Deflection for X-beams with RCA during loading were measured and compared to the standard concrete, see Figure 8-8 for RCCA and Figure 8-10 for RCA. The theoretical values for deflection (δ mm) were calculated as follows:

Equation 8-2 $$\delta = \frac{P\,L^3}{48\,E\,I} = \frac{M\,L^2}{12\,E\,I}$$

P is the load (N), *M* is the bending moment (Nmm), *E* is the concrete modulus of elasticity measured using resonant frequency (*ERUDITE*) and pulse velocity (*PUNDIT*) methods (32900 N/mm^2 for NA and 27900 N/mm^2 for RCA), see Appendix 4 for details. I is uncracked second moment of area for the X-beam, see Figure 8-1, (I = 136587661 mm^4). For calculation details of I cracked value see Appendix 4.

Deflections of X-beams with 20% RCCA are similar to that of standard concrete. However at 50% RCCA, deflections are greater than that of standard concrete and even greater than the theoretical values of standard concrete but lower than that of RCA theoretical values, see Figure 8-8.

When combining RCCA & RCFA (RCA) deflection in both 20% and 50% replacements beams are greater than that of the standard concrete and the theoretical values of standard concrete but almost match the RCA theoretical values, see Figure 8-10 and Figure 8-11. The effects in the deflection were clearly revealed via the occurrence of the cracks. RCA X-beams propagated cracks at lower load than the RCCA X-beam and the standard X-beam; see Section 8.5. Unlike the deflection, the concrete strain did not show clear differences when using RCCA this will be discussed in the next section.

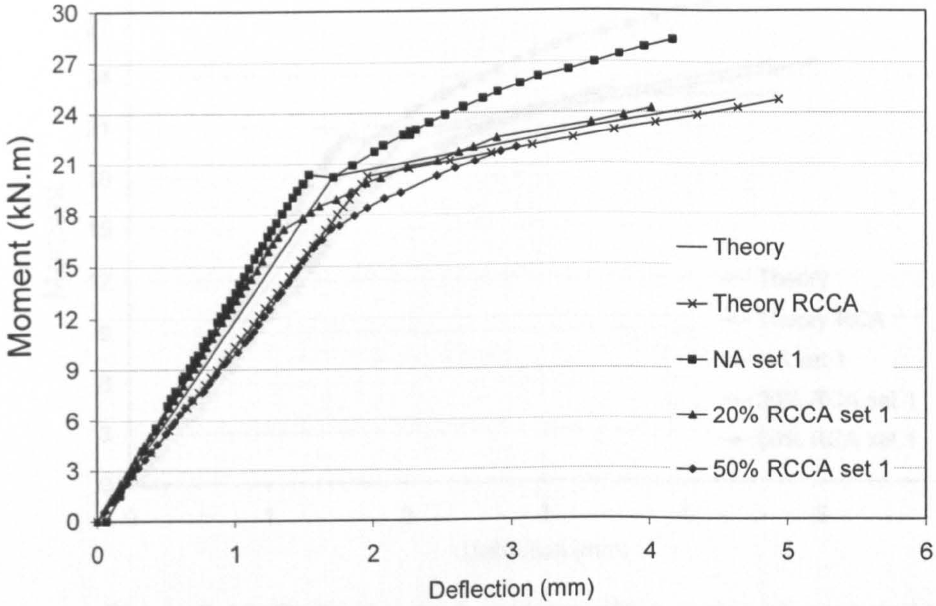


Figure 8-8: Deflection with different RCCA replacements (set 1)

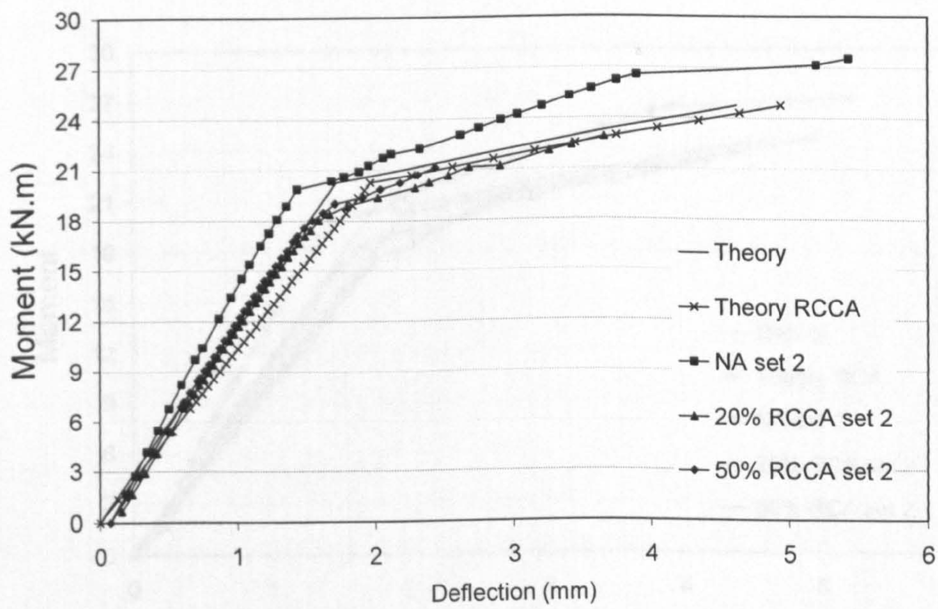


Figure 8-9: Deflection with different RCCA replacements (set 2)

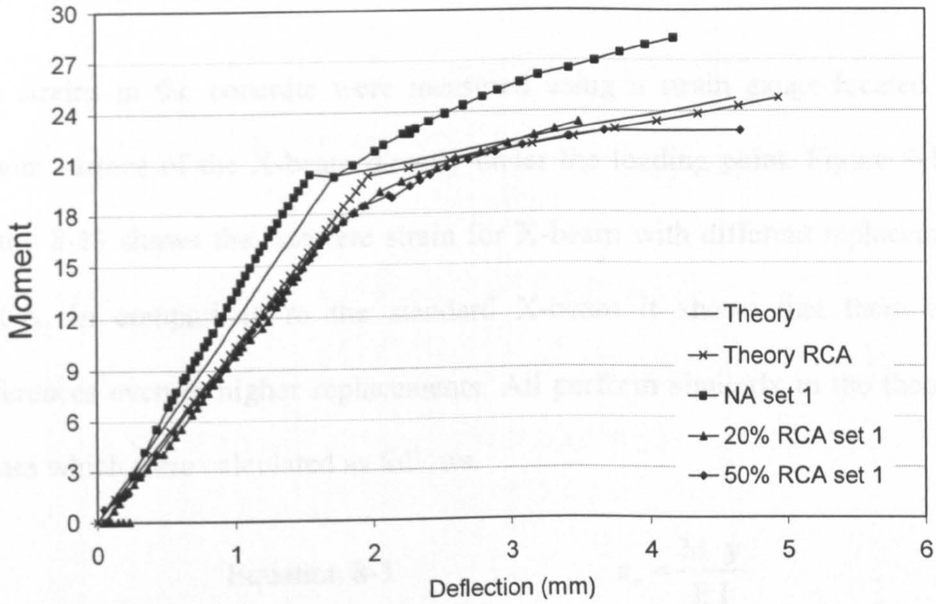


Figure 8-10: Deflection with different RCA replacements (set 1)

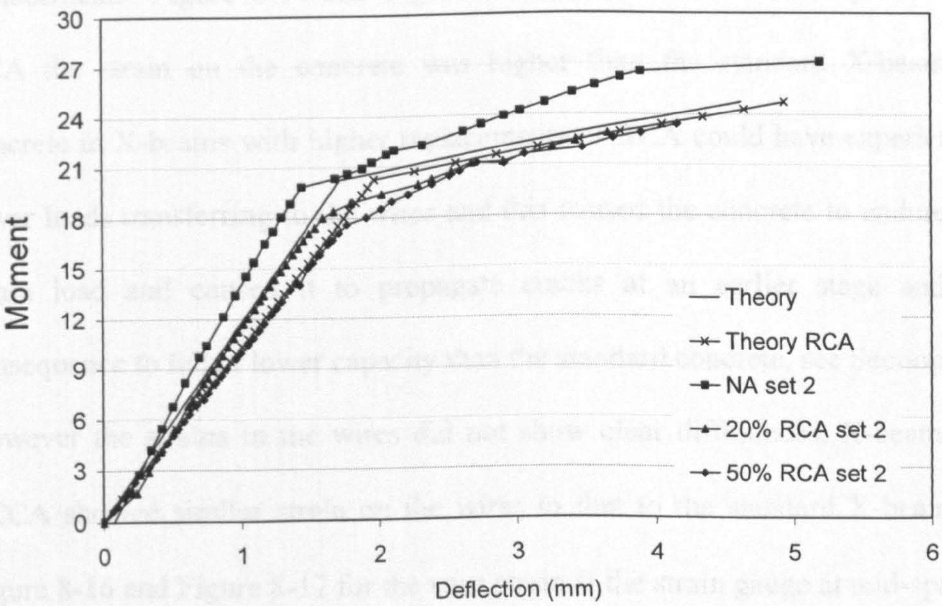


Figure 8-11: Deflection with different RCA replacements (set 2)

8.6.2 Strain in the Concrete and the Prestressing Wires

The strains in the concrete were measured using a strain gauge located at the bottom surface of the X-beam directly under the loading point. Figure 8-12 and Figure 8-13 shows the concrete strain for X-beam with different replacement of RCCA. In comparison to the standard X-beam it shows that there are no differences even at higher replacements. All perform similarly to the theoretical values which were calculated as follows.

Equation 8-3 $$\varepsilon_c = \frac{M\ y}{E\,I}$$

ε_c is the concrete strain, M is the applied moment (Nmm), y is the distance to the centroidal axis (y = 112.87 mm), and E and I as before.

When combining RCCA and RCFA some differences start to reveal at higher replacements. Figure 8-14 and Figure 8-15 shows that at 50% replacement of RCA the strain on the concrete was higher than the standard X-beam. The concrete in X-beams with higher replacements of RCA could have experienced a lower loads transferring to the wires and this caused the concrete to endure more strain load and caused it to propagate cracks at an earlier stage and as a consequence to fail at lower capacity than the standard concrete, see Section 8.5.

However the strains in the wires did not show clear differences. X-beams with RCCA showed similar strain on the wires to that to the standard X-beams, see Figure 8-16 and Figure 8-17 for the wire strain at the strain gauge at mid-span, see Appendix 5 for all the figures to all the strain gauge data.

Nevertheless, standard X-beams and that with RCCA both reported to have lower wire strains than the theoretical values, which was calculated as the concrete strain using Equation 8-3 where y = 77.8 mm is the distance between the centroidal axis to the wires, see Figure 8-1. Similarly for X-beams with RCA, they did not show clear differences comparing to the standard X-beam but showed again lower wires strain than the theory see Figure 8-18, all figures in Appendix 5.

Wires strain measurement in these tests did not expose the effects of RCA on the performance of the X-beams but the cracks clearly propagated at lower moments and X-beam failed at lower capacity when using higher replacements of RCA as was seen in Section 8.5, more will be discussed in section 8.7.

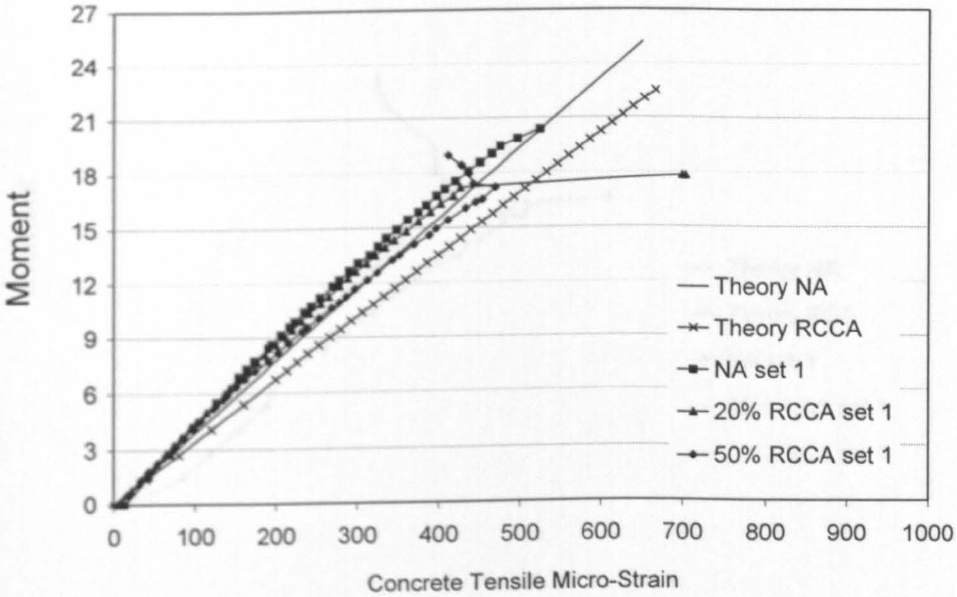


Figure 8-12: Concrete strain for X-beam with RCCA during the test (set 1)

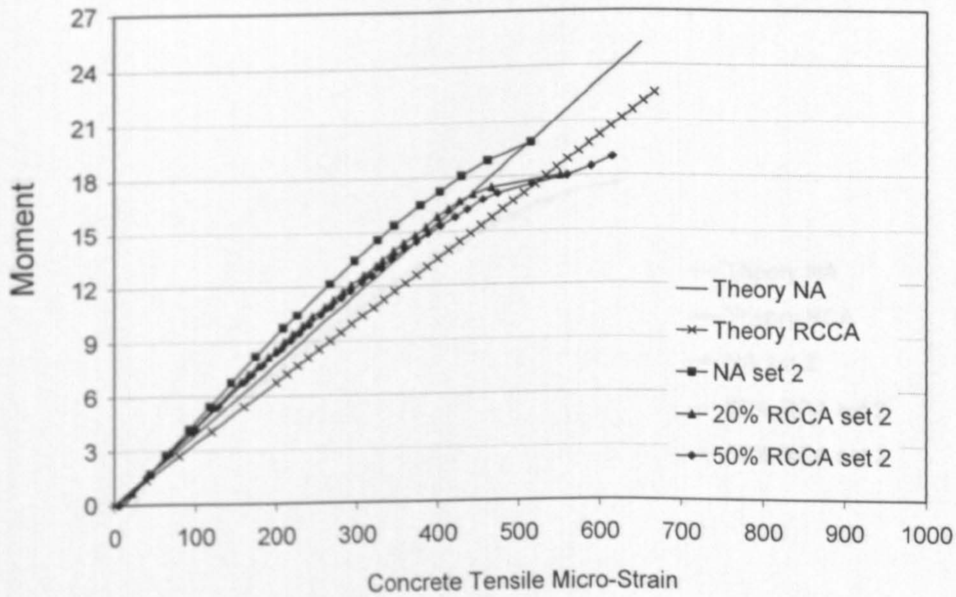


Figure 8-13: Concrete strain for X-beam with RCCA during the test (set 2)

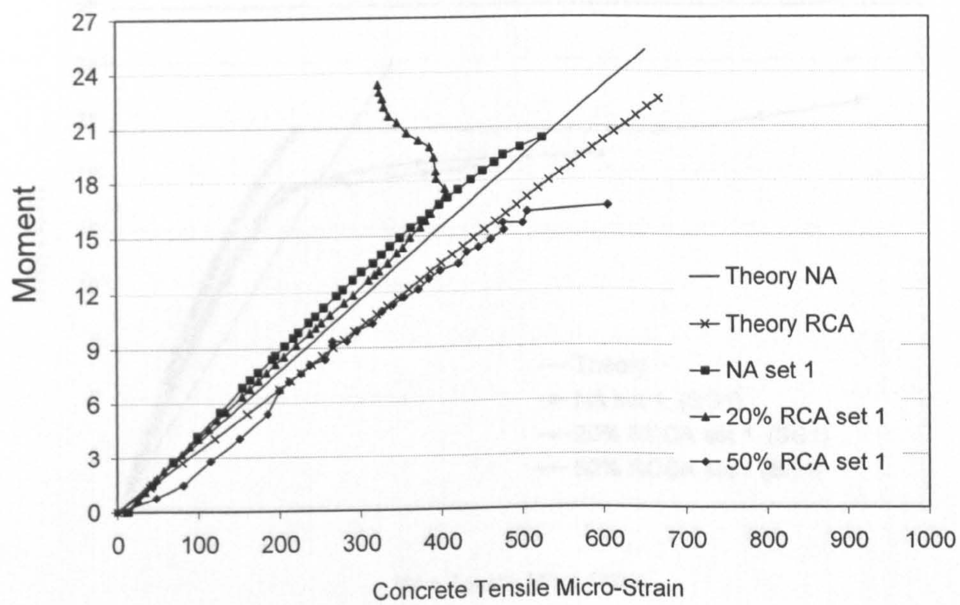


Figure 8-14: Concrete strain for X-beam with RCA during the test (set 1)

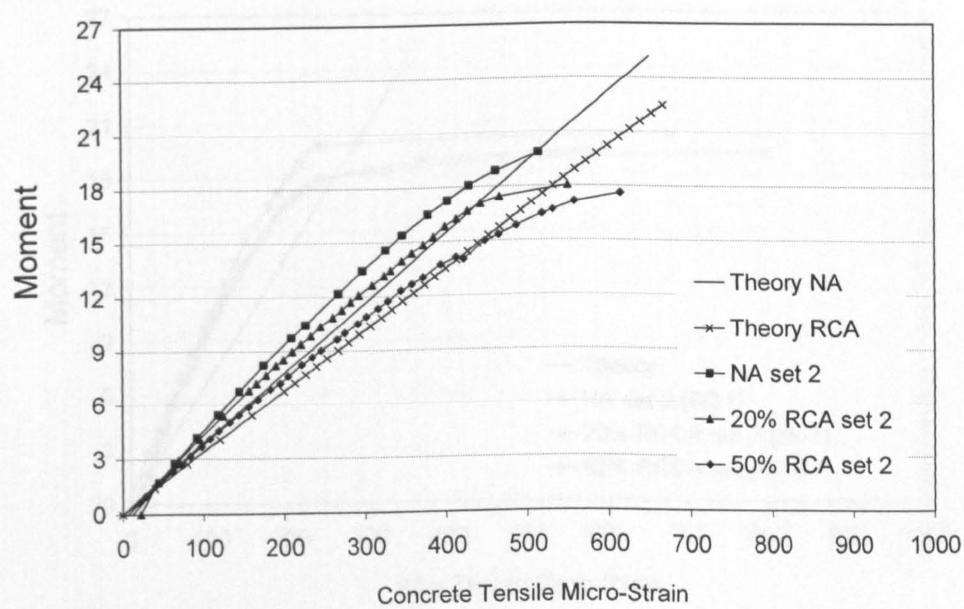


Figure 8-15: Concrete strain for X-beam with RCA during the test (set 2)

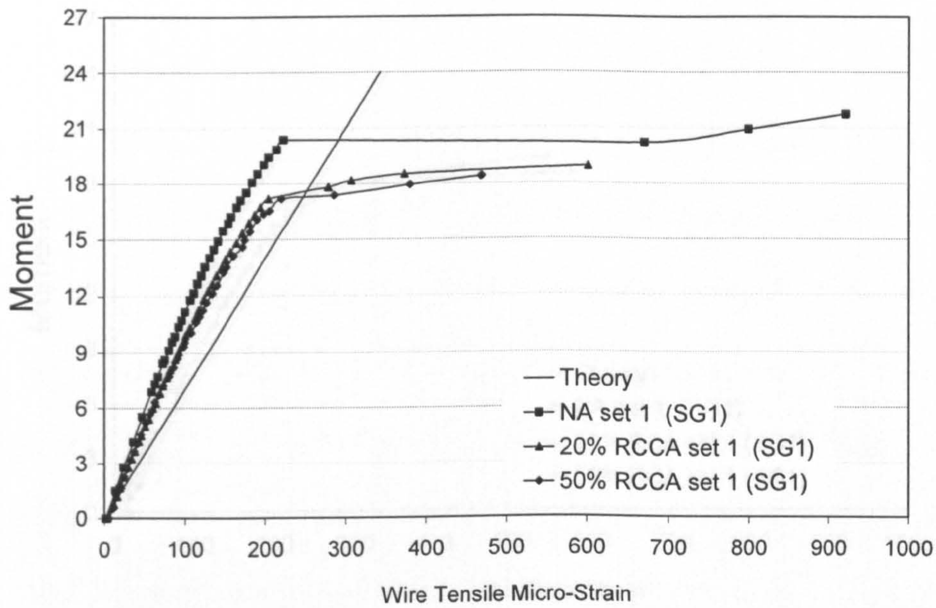


Figure 8-16: Wire strain at SG1 for X-beam with RCCA during the test (set 1)

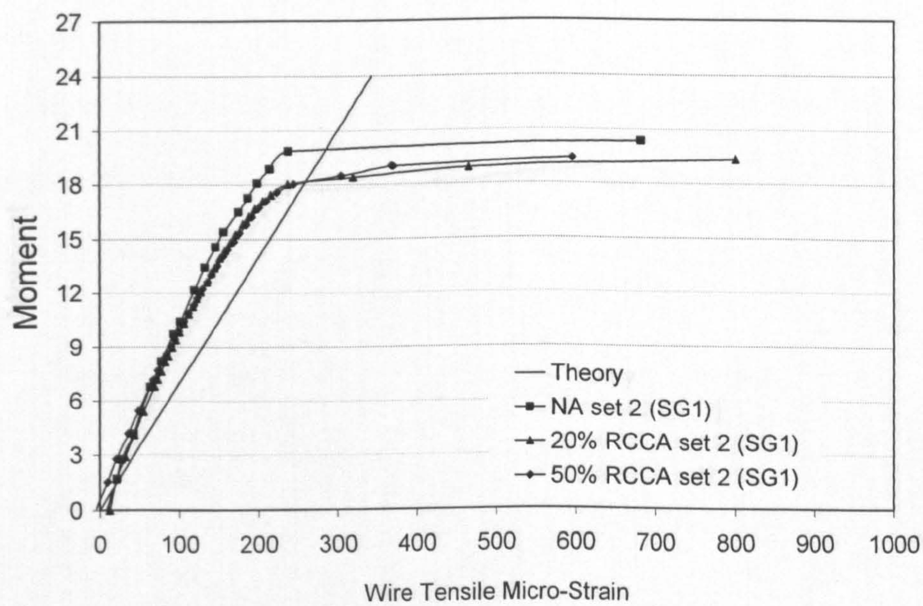


Figure 8-17: Wire strain at SG1 for X-beam with RCCA during the test (set 2)

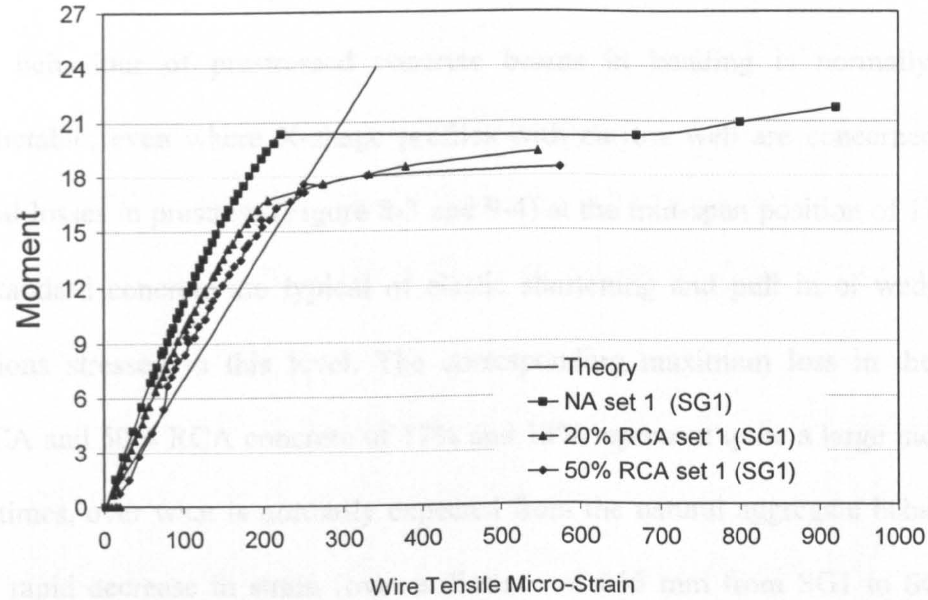


Figure 8-18: Wire strain at SG1 for X-beam with RCA during the test (set 1)

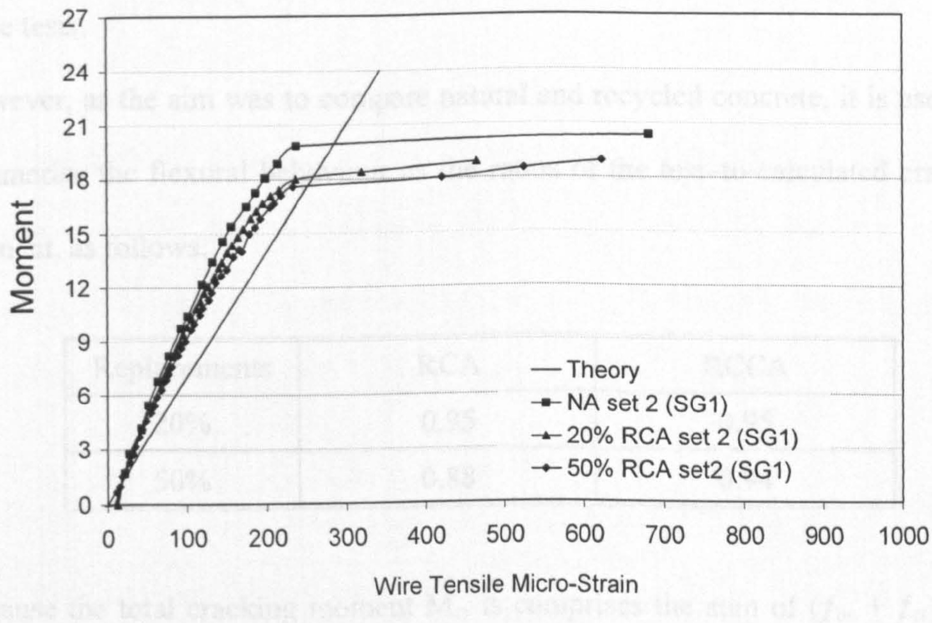


Figure 8-19: Wire strain at SG1 for X-beam with RCA during the test (set 2)

8.7 Discussion

The behaviour of prestressed concrete beams in bending is normally very predictable, even where X-shape profiles with narrow web are concerned. The initial losses in prestress (Figure 8-3 and 9-4) at the mid-span position of 11-13% in standard concrete are typical of elastic shortening and pull in of wedges in sections stressed to this level. The corresponding maximum loss in the 50% RCCA and 50% RCA concrete of 17% and 18% represent quite a large increase, 1½ times, over what is normally expected from the natural aggregate behaviour. The rapid decrease in strain (over a distance of 666 mm from SG1 to SG3) in these specimens reflects some of the poor bond behaviour reported in Chapter 7. It is clear that the wires have not been permitted to attain their full transmission (development) length, and this must be seen as a short-coming in the design of these tests.

However, as the aim was to compare natural and recycled concrete, it is useful to summarise the flexural behaviour as the ratios of the test–to-calculated cracking moment, as follows:

Replacements	RCA	RCCA
20%	0.95	0.95
50%	0.88	0.94

Because the total cracking moment M_{cr} is comprises the sum of $(f_{bc} + f_{ct})\,Z_b$, a reduction in M_{cr} of 12% (for 50% RCA) actually represents 16% reduction in f_{bc}. (f_{ct} is flexural tensile strength of the concrete). The corresponding reductions in f_{bc} in the 20% RCA and 20% RCCA is only 7%.

A recommendation could therefore be made that the effective prestress is taken as 0.93 f_{bc} in the case of 20% replacements but no further recommendation can be made for higher replacement without further testing. The development of both flexural and shear-tension cracking was a positive feature of these tests, observed for both natural and recycled aggregate concrete, although the lack of ductility experienced in the final shear failure was not.

However previous testing of X-beams by Paine [87] and various hollow core slab shear tests reported by Elliott [94] suggest that these features were not unusual. The rapid release in energy along the sheared plane caused fractured surfaces to be quite smooth (to the touch). Given that the ultimate moment of resistance was not achieved (owing to the shear cracks), it is not useful to discuss the ultimate data, except to report that the ratio of the ultimate test moment for the recycled-to-natural aggregate beams was 0.77 to 0.83, with no particular trend evident.

The variations in flexural rigidity, as determined from plots of moment *vs* deflection and moment *vs* surface tensile strain, were less consistent than the variations in moments. The ratios of the flexural rigidity (*EI*) for the recycled –to natural aggregate beams, up to point of cracking, are as follows:

	RCA		RCCA	
Replacements	Deflection	Strain	Deflection	Strain
20%	0.81	0.95	0.91	0.92
50%	0.76	0.80	0.88	0.98

The RCCA clearly out performed the RCA (by up to 18 percentage points). Given that the variation in the second moment of area is small, these changes must be attributed to variations in Young's modulus, the effect of tension stiffening in the cracked section, and on the integration of the cracked section over the full span of the beam.

The results for the strains (SG1) at mid-span in the top wire show similar trends, but are considerably lower than the theoretical value, typically 20% lower, given that the surface strains were in very good agreement with theoretical values, the simple elastic Equation 8-3 is adequate. Therefore the reduced wire strains must reflect a loss in bond immediately upon loading.

The recommendation based on deflections and surface strains is as follows. Although the 50% RCA data appear to be poor (reduction in EI to 0.76), in the context of the absolute deflection before cracking being less than span / 900 (i.e. 2 mm in 1800) and the concrete achieving an imposed stress of about 20 N/mm^2 (i.e. 600 x 10^{-6} strain), there is no evidence to deny the use of all mixes in this context.

8.8 Conclusion

Prestressed X-shape beams were used to assess the effects of using of RCA on the flexural and shear behaviour. The reason for choosing an X-shape is because it closely simulates the rounded webs of an extruded hollow core slab units, see Figure 8-1. Three point loading tests were used, see Figure 8-2.

It is concluded that using 20% replacements of RCCA did not largely affect the X-beam flexural behaviour. For 20% RCCA replacements, the prestressing loss

(Table 8-3), deflection and flexure crack failure were similar to the standard X-beam and within the design limit.

However at higher replacements (50%) some deterioration starts to reveal, as initially reported by the prestress loss, then by the deflection and crack development and finally failure, see Table 8-4. The effects are even greater when using a combination of RCCA and RCFA.

The maximum loss of prestress at the mid-span position in the 50% RCCA and 50% RCA concrete of 17% and 18% represent quite a large increase, 1½ times, what is normally expected from the standard concrete (11-13%). The rapid decrease in strain (over a distance of 666 mm from SG1 to SG3) in these specimens reflects the poor bond behaviour. The ratios of the flexural rigidity (EI) for the recycled-to-natural aggregate beams, up to point of cracking showed that the RCCA clearly out perform the RCA (by up to 18 percentage points).

Therefore using a combination of RCCA and RCFA could cause a poorer behaviour for prestress unit of this type. 20% of RCCA showed little effect on the prestress concrete performance. However if higher replacements proportions of RCCA were used then lower prestressed concrete design capacity should be altered accordingly. Using 50% of RCCA caused a failure around 20% lower than the standard X-beam and 25% lower than the calculated ultimate failure.

9 Summary of Conclusions and Recommendations

9.1 Conclusions

Many governments, including Britain, have recognised the importance of sustainable development and made it one of its central themes of their economic and social programmes. Among these programmes, construction has been recognized as one of the main priorities because, apart from the fact of its impact on the economy, it is known to be the largest single source of waste and for those reasons many government's policies have been set out to develop a more sustainable future. One of the objectives was the prudent use of natural resources and to achieve this it is believed that sustainable construction should encompass many issues such as the reuse of existing built assets, designing for minimum waste and maximum building flexibility and to minimize resources and energy usage.

The challenge for the construction industry is now to move towards environmentally responsible policies while maintaining the economic viability. Many leading companies are considering such challenges and in doing so are creating new markets and opportunities to meet the needs of a rapidly changing environment. To encourage specialist companies, mainly precast concrete firms, to meet their challenges and obligations this research work was aimed to study the properties of recycled concrete derived from hardened prestressed hollow core slab units (hcu) crushed to –14 mm using three different methods - cone, jaw and impact crushers.

Commercial hcus, manufactured (by Richard Lees Ltd) using the Spiroll extrusion technique, were obtained for this study. The recycled material was separated into fractions of 14 mm, 10 mm and –5 mm, and tested for physical and mechanical properties relevant to use in concrete. Concrete was then made using certain proportions replacement of recycled coarse (RCCA), recycled fine (RCFA) and mixed (RCCA+RCFA) aggregates. The control mix was made using natural limestone coarse and natural fine aggregates. The following are the main conclusions that evolved through out this work:

9.2 Crushing Machines and Characteristics of RCA

Different crushing methods have an influence on the properties of recycled concrete aggregate. Physical properties, specifically shape and texture, appear to be affected the most and showed some variance related to each type of crusher. The impact crusher appears to be the most suitable overall by producing RCA with better shape and strength. This is followed by the jaw crusher and then the cone crusher. There is no distinctive influence of the crushing methods on the water absorption and density for RCA. Comparing the coarse RCCA with natural limestone aggregate, the RCCA showed similar (or even better) properties especially for shape and strength. However, the RCCA will absorb about 4 to 6 times more water than the natural limestone aggregate; thus it is recommended to pre-wet the RCA with some of the mixing water before mixing.

All RCCA appear to have a reasonable grading comparing with the British Standard and is similar to that for natural limestone aggregate. It should be noted that the impact crusher has one major disadvantage, which is that it has a large reduction factor (from the feed to the output) and consequently it produces large amount of fine aggregate than coarse aggregate. The fine RCFA was considerably coarser than the natural river gravel, and technically did not comply with the BS coarse category limits, failing at the 2.36mm sieve size only. The effect of a smaller fraction of RCFA below 600 μm may have a significant effect on the desired mix proportions to keep workability constant. Although one crusher performed well in some properties and showed some disadvantages in others, all produced recycled concrete aggregate with acceptable strength and shape (Table 4-1 and Table 4-6).

9.3 Concrete with Recycled Concrete Aggregate

9.3.1 Workability

The workability is very important to the commercial production of extruded or slip-formed hollow core slab units. Manufacturers are careful to control workability by controlling water content, allowing the strength of concrete to fluctuate if the workability has to be adjusted.

The slump value of fresh concrete made with RCA varied widely depending on the percentage and type of replacement, and the type of crusher, a fact which may be linked to the angularity of the RCCA. The compaction factor of fresh concrete made with RCA was more consistent and logical. Nevertheless there are many factors affecting the workability (e.g. water content, absorption rate, shape and surface texture, and fine content) and are not independent of each other in their effects on concrete workability

Replacement up to 50% of RCCA did not significantly affect the workability. However, using up to 20% of RCFA causes the workability to increase and that should be related to both extra water and the finer elements in RCFA that caused lubrication which may be interpreted as higher workability. Combining the RCFA and RCCA reduced the workability more so with higher replacement percentages.

Slump and compacting factor patterns are fairly consistent; that for a fixed aggregate / cement ratio of 4.5 the slump decreases as the compacting factor decreases, this agrees with Neville [59] as shown in Figure 5-9.

The conclusion is that it is necessary to follow a technique of pre-wetting the RCA with some of the mixing water in the mixer pan to enable it to absorb as much of the extra water and to prevent a rapid decrease in workability.

9.3.2 Compressive and Flexural Strength

Concrete with RCCA for 20% and 50% proportions show better cube compressive strength than the control mix, but concrete with similar proportion of RCFA reduced the compressive strength. When combining RCCA and RCFA together (with similar proportions) the concrete compressive cube strength remained similar to that for the control mix. This concludes that using those proportions of RCA (derived from hcu) can be used to produce concrete with high compressive strength.

For tensile strength, RCCA (20% and 50%) showed better performance than that of natural limestone. Clearly, these recycled aggregates have bonded well with the new cement paste and the flexural failure just went through these aggregates and caused higher value for the tensile strength. There were similar findings for RCFA and a combination of RCCA and RCFA.

The relationship between the design principle flexural (and tensile) strength and compressive strength defined in BS 8110 can be used for concrete with RCA derived from hcu.

The SS density of concrete with RCCA and with RCFA is lower than that of the control concrete and reduces further as the replacement increased. Higher reductions occurred when combining RCCA and RCFA together as this is because of the added extra water to compensate for absorption, as well as other factors like bleeding. It is valid to say that the strength of concrete depends on the aggregates used, but it is equally correct to say such strength cannot be related to any single physical characteristic.

The differences in aggregates characteristics from different crushes showed no significant difference in concrete strength and thus any of them could be used for production of recycled aggregates.

9.4 Bonding Between RCA Concrete and Reinforcement

Two methods were used to interpret the data obtained from prismatic bar bonding tests comprising of re-bars and prestressing wires. The first was based on the crack patterns and the average distance between the cracks using the bar friction coefficient values. The second was using the strain energy obtained by plotting the tension against the elongation.

Reinforcing bars:

For reinforcing bars it could be concluded that using a high proportions of RCA would reduce the bonding strength between the concrete and the reinforcing bars, although some tests (based on the strain energy ratio) showed better bonding when using RCA.

In mortars most tests reported a lower bonding strength with high proportions of RCFA. Although in some tests (using the friction coefficient value) the RCFA showed slightly better bonding with reinforcing bars. Thus it is recommended to use a moderate proportion of RCCA, up to 20% and preferably not combining RCCA and RCFA for concrete applications where the bonding with the reinforcements is a critical concern.

Pretensioning wires:

In pretensioning wires the RCA performs better in bonding than that with the reinforcing bars but still not as good as that the natural aggregate. The strain energy ratio showed a poorer performance in bonding when using 100% RCA in comparison to that of the standard concrete. In mortars most methods concluded that high proportions of RCFA cause deterioration in bonding strength with wires, and it was not as good as standard mortar. Thus for prestressing concrete where the bonding with the tendons is a critical concern it is recommended to use a moderate proportion of RCCA, up to 20% and preferably not combining RCCA and RCFA.

9.5 Flexural Behaviour of Prestressed X- Beams with RCA.

Prestressed X-shape beams, which simulate the rounded webs of an extruded hollow core slab units, were used to assess the effects of using RCA on flexural behaviour. It is concluded that using 20% replacements of RCCA would not affect the X-beam flexural behaviour. The prestressing loss, deflection and X-beam flexure crack failure were similar to the standard X-beam and within the design limit. Deterioration will occur at higher replacements (50%). Initially by the prestress loss then by the deflection and then crack developments and failure, see Table 8-4. The deterioration are even greater when using a combination of RCCA and RCFA. The maximum loss of prestress force in the tendons at the mid-span position in the 50% RCCA and 50% RCA is 1½ times of what is normally expected from the standard concrete. The rapid decrease in strain mid span to ends reflects the poor bond behaviour. The reduced wire strains reported in the tests must reflect a loss in bond immediately upon loading.

Method set by BS8110 to calculate the deflection can be used for RCA concrete; test results showed consistency between theory values and tests measured values. Elastic Equation $\varepsilon_c = \frac{M\ y}{E\ I}$ is found to be adequate for concrete with RCA as the surface strains were in very good agreement with theoretical values. The recommendation based on deflections and surface strains is as follows, in the context of the absolute deflection before cracking being less than span / 900 (i.e. 2 mm in 1800) and the concrete achieving an imposed stress of about 20 N/mm2 (i.e. 600 x 10-6 strain), there is no evidence to deny the use of all mixes in this context.

The effective prestress in the case of 20% replacements is recommended to be taken as 0.93 f_{bc}. Test result found a 5% reduction of Mcr (for 20% RCA) which represents 7% reduction in f_{bc}, this is because the total cracking moment M_{cr} is comprises the sum of $(f_{bc} + f_{ct}) Z_b$.

9.6 Recommendation for Further Work

1. During the production of RCA it was experienced that a sufficient amount of RCFA (below sieve 5 mm) was produced and throughout the research work it was found that RCFA considered as one of the main causes for some deterioration effects in the concrete properties. To minimise such undesirable effects and the effort and cost of re-screening the RCFA, it would be required to identify what lower sieve size of RCFA could be used and what lower sizes (that contribute to undesirable effects) could be excluded from concrete production.

2. One of the main common factors that negatively affect the concrete behaviour while using RCA is the added excess total water. This is because of the higher RCA water absorption, thought to be the results of cement paste and/or mortar attached to the recycled aggregates, although this was not actually measured in this research. Minimising the amount of such attached material will greatly improve the RCA concrete behaviour. Achieving this will require further studies (specific research) in the mechanism of the crushing machines to develop and adapt them to be more suitable for the production of cleaner RCA. It should be noted that most the existing crushes were manufactured for the purpose of natural quarries, and not concrete crushing.

3. Because of the shape and texture of RCA and the added extra water, the existing method of measuring the concrete workability (though it did show guidance) struggled to give accurate values that reflected the actual workability and feasibility of handling fresh RCA concrete. Further studies should be useful to develop some other methods with acceptable sensitivity that will be used for RCA concrete.

4. In this research a prestressed X-shape beams (simulates the rounded webs of an extruded hollow core slab units) were used to assess the effects of using of RCA on the performance (flexural behaviour mainly) of hollow core slab units. Shear-flexural crack types were encountered during the tests so It will be recommended to do further studies to clarify the shear capacity and shear failure mechanisms of hcu's with RCA. To complement the research work further it will be necessary to carry out full-scale shear and flexural tests on a factory cast hollow core slab units with RCA.

Appendix 1

1. Questionnaire

Please indicate your level of priority for the following subject areas and if you think you need to ***add any other areas please do not hesitate do add it*** on the next page.

	High	Med.	Low
Crushing Methods (e.g. developing procedure to assess correct degree of crushing)			
Grading (e.g. effect of size distribution on performance; fines, gap grading)			
Shape (e.g. influence of Flaky or very angular shapes)			
Texture (e.g. effect of fractured matrix of recycled materials)			
Water Absorption (e.g. effect of mix design and workability)			
Impurities (e.g. poor quality original materials or chemically attacked material in recycled concrete)			
Durability (e.g. effect of re-introduction of additional cement; less durable recycled material)			
Alkalis (e.g. excessive alkalis)			
Latent Hydration (e.g. benefits if unhydrated cement present in recycled material; strength gain; heat of hydration)			
Mixing (e.g. mixing periods or methods affected)			
Workability (e.g. modified workability characteristics; new methods of measurements)			
Compaction (e.g. effect on different compaction techniques; uniformity of mix; standard deviation)			
Water bleeding (e.g. additional measures taken to monitor or prevent it)			
Segregation (e.g. effect of recycled material on compacting efforts)			
Shrinkage (e.g. effect of reintroduction of cement as part of aggregate)			
Cracking (e.g. plastic settlement; shrinkage; crack initiation propagation)			

Creep (e.g. effect of additional cement)			
Admixtures (e.g. interaction with admixture; effect of admixtures in recycled materials)			
Strength			
	High	**Med.**	**Low**
Compression			
Tensile			
Bond			
Modulus of Elasticity			
	High	**Med.**	**Low**
Static			
Dynamics			
Poisson's Ratio			

Any other subject areas might be of interest or any comments ;
Would you please Write it down

Appendix 2

1. Estimation of the amount of water that absorbed by the oven dry aggregate during the mixing: Approach-A, Approach-B and Approach-C
2. Aggregates Tests (AIV, TFV, FI and AN) mentioned in Chapter 4
3. Slump and Compacting Factor Workability for tests reported in Chapter 5
4. Mean of Compressive Cube, Tensile Splitting Strength and Flexural Strength and Standard Deviation (N/mm^2) for specimens reported in Chapter 5
5. S. S. Density mean (prisms) and Standard Deviation (Kg/m^3) for specimens reported in Chapter 5
6. Compressive Cube, Tensile Splitting Strength and Flexural Strength (N/mm^2) for specimens reported in Chapter 5 (for all crushers)

Estimation of the amount of water that absorbed by the oven dry aggregate during the mixing. (Approach A)

Mix Procedure

- Weight the aggregates (mix proportion in the provided table)
- Weight the glass container
- Put the aggregate in the glass container and Mix the aggregate probably.
- Add the mixing water and keep mixing the aggregate until all are wet.
- Leave the aggregate covered to absorb the water for 10 minutes
- Wet the sieve slightly and then weight it.
- Pour the aggregate carefully on the sieve.
- Weight the glass again (after emptying the aggregates)
- Leave the aggregate on the sieve to drain.
- Then weight the sieve with the aggregates.
- Put the aggregate in the oven and dry them for 24hrs
- Weight the oven dried aggregate again
- Then weight the sieve alone

The following tables show the test results.

Results of Approach A tests

NA 100 %

Mix one	
14 mm (Limestone)	2040
10 mm (Limestone)	1355
Sand (gravel sand)	2150
Water	705

		Mix 1-B	Mix 1-C
A	water added (from table)	705	705
B	weight of all aggregate (from table)	5545	5545
C	Weight the glass container	6766	6766
D	Weight of the sieve slightly wet.	973.5	972.5
E	Weight the glass again (after emptying the aggregates)	6766.5	6769.5
F	Then weight the sieve with the aggregates.	6944.5	7052
G	Weight of oven dried aggregate again + Sieve	6493	6494
H	Weight the sieve alone.	965	963
K	Aggregate wet = (weight of Agg + sieve) - sieve	5971	6079.5
L	Agg Oven Dry after mixing (G – H)	5528	5531
M	Agg Loss = (B - X)	17	14
N	Agg Loss in Glass = E – C	0.5	3.5
Q	Water Absorbed = (K- L - N)	442.5	545
R	Actual % of water absorbed = (Q/A) %	63%	77%

Results of Approach A tests

RCA 100 %

Mix Two	
14 mm RCA	1962
10 mm RCA	1300
Sand RCA	2064
Water	924

		Mix 1-A	Mix 1-B	Mix 1-C
A	Water added (from table)	924	924	924
B	Weight all aggregate (from table)	5326	5326	5326
C	Weight the glass container	6766	6766	6766
D	Weight of the sieve slightly wet.	982	981.5	980.5
E	Weight the glass again (after emptying the aggregates)	6769	6769.5	6767
F	Then weight the sieve with the aggregates.	7101	7007.5	7041
G	Weight of oven dried aggregate again + Sieve	6239	6003.5	6211.5
H	Weight the sieve alone.	964.5	965	965
K	Aggregate wet = (weight of Agg + sieve) - sieve	6119	6026	6060.5
L	Agg Oven Dry after mixing (G - H)	5274.5	5038.5	5246.5
M	Agg Loss = (B - X)	51.5	287.5	79.5
N	Agg Loss in Glass = E – C	3	3.5	1
Q	Water Absorbed = (K- L - N)	841.5	984	813
R	Actual % of water absorbed = (Q/A) %	**91%**	**106%**	**88%**

Approach - B Tests

WATER ABSORPTION TESTS

(50% RCA)		(50% RCCA) 10 mm	
Aggregate size	Kg	Aggregate size	Kg
14	1.131	10 mm	6164
10	0.754		
Sand	1.197		
14 RCCA	1.131		
10 RCCA	0.754		
RCFA	1.197		

Test A: 24 hours soaking

Weigh all the aggregate above, mix them dry and put them all in one container.
Then follow the British standard for doing water absorption (i.e. **soaking** -fully immersed in water **for 24** hours and Oven dry 24hrs.... etc)

Test B: 10 minutes soaking

Weigh all the aggregate above, mix them dry and put them all in one container.
Then follow the British Standard for doing water absorption BUT we need to soak-fully immersed in water only for 10 Minutes NOT 24 hrs. While Oven dry remain the same for 24hrs.

Results approach-B

Aggregates W/A after 10minuts and 24 hours soaking

Aggregates	Soaking time	W/A
All aggregates (Mixed of all sizes) 50% RCA	10 minutes	4.8%
	24 hours	32.4%
Coarse Aggregates (10 mm) (50% RCA)	10 minutes	2.8%
	24 hours	4.5%

Approach – C

Difference between using oven dry aggregates to that with 24 soaking

Set one Table A (mix as in the thesis)

- Use mix proportions in table A
- The aggregate is weighed and mixed dry for about 30 seconds.
- One-third of the water should be added and the mixing continued for 2 minutes.
- Then the aggregates were covered to stand still for almost 10 minutes.
- After that the cement was added and mixed for 30 seconds and then the remaining water was added and the mixing continued for another 2 minutes.
- Then do specimens as usual

Set two Table B (soaking for 24 hours)

- Use mix proportions in Table B
- Weighed the aggregate only as in table B and then soak them in water for 24hrs
- Remove the aggregate from the water. The aggregate should be in SSD conditions and it was difficult to make sure no excess/floating water is attached to the aggregates (following the British standard in aggregate density sections which explains how the SSD aggregates should appear)
- Mix the aggregates about 30 seconds. Follow mixing as usual
- Then do specimens as usual

Table A

Material	Mix Proportion in weight from a total of 20 kg	
	NA	**100%**
O.P. Cement	**3.34**	**3.34**
Water	**1.87**	**2.59**
Sand*	**5.74**	**-**
10 mm*	**3.61**	**-**
14mm*	**5.44**	**-**
RCFA*	**-**	**5.50**
10 mm RCCA*	**-**	**3.41**
14mm RCCA*	**-**	**5.16**

*Aggregate in OD

Table B

Material	Mix Proportion in weight from a total of 20 kg	
	NA	**100%**
O.P. Cement	**3.34**	**3.34**
Water	**1.66**	**1.66**
Sand*	**5.74**	**-**
10 mm*	**3.61**	**-**
14mm*	**5.44**	**-**
RCFA*	**-**	**5.50**
10 mm RCCA*	**-**	**3.41**
14mm RCCA*	**-**	**5.16**

Result for Approach-C

100% RCA

Average values

100% RCA	7 days N/mm2	kg/m3
Normal	26	2236
Soaking	31	2247

100% RCA	28 days N/mm2	kg/m3
Normal	35	2246
Soaking	41	2247

Details

100% RCA Compressive strength

Normal Mixing			
7 days	26.2	25.2 26.1	26.9
28 days	35.2	35.0 34.8	34.2

Soaking			
7 days	30.4	30.4 30.6	31.0
28 days	41.8	41.8 41.3	40.2

100% RCA Density strength

Normal Mixing			
7 days	2232	2242 2236	2235
28 days	2248	2256 2247	2238

Soaking			
7 days	2244	2247 2246	2248
28 days	2241	2248 2247	2253

NA

Average

NA 7 days

	N/mm2	kg/m3
Normal	41	2417
Soaking	34	2402

NA 28 days

	N/mm2	kg/m3
Normal	47	2410
Soaking	42	2397

Details

NA

Compressive strength

Normal Mixing			
7 days	40.8	40.8	40.4
		40.67	
28 days	48.0	46.2	46.4
		46.9	

Soaking			
7 days	34.8	32.6	34.4
		33.9	
28 days	41.4	41.6	41.7
		41.6	

NA

Density strength

Normal Mixing			
7 days	2419	2417	2415
		2417	
28 days	2408	2413	2410
		2410	

Soaking			
7 days	2411	2390	2405
		2402	
28 days	2403	2394	2395
		2397	

Aggregates properties from different crushers

Cone Crusher

A.I.V	Cone
S1	25.5
S2	27
S3	23.2
A.I.V	**25**
	25

TFV	Cone
S1	96
S2	128
S3	107
TFV	**110**
	110

F.I	Cone
S1	21%
S2	20%
S3	-
F.I	**21%**
	21

AN	Cone
S1	11
S2	11
S3	-
AN	**11**
	11

14 mm RCCA			
Wt (gm)	S1	S2	**Avg**
B	2740	2741	
C	2100	2100	
A	1016	1018	
D	973	975	
W.A	4.4%	4.4%	**4.4%**
Density (SSD)	2.702	2.700	**2.701**
Density (O.Dry)	2.588	2.586	**2.587**

10 mm RCCA			
Wt (gm)	S1	S2	**Avg**
B	2672	2671	
C	2069	2069	
A	1020	1018	
D	975	973	
W.A	4.6%	4.6%	**4.6%**
Density (SSD)	2.446	2.447	2.446
Density (O.Dry)	2.338	2.339	2.338

Fine Aggregate RCFA			
Wt (gm)	S1	S2	**Avg**
B	2362	2362	
C	2068	2068	
A	506	506	
D	474	474	
W.A	6.8%	6.8%	**6.8%**
Density (SSD)	2.387	2.387	**2.387**
Density (O.Dry)	2.236	2.236	**2.236**

Impact Crusher

A.I.V	
S1	22.7
S2	24.1
S3	-
A.I.V	**23**
	23

TFV	
S1	174
S2	165
S3	-
TFV	**169**
	170

F.I	
S1	9%
S2	9%
S3	-
F.I	**9 %**
	9%

AN	
S1	9
S2	9
S3	-
AN	**9**
	9

14 mm RCCA			
Wt (gm)	S1	S2	**Avg**
B	2676	2676	
C	2069	2069	
A	1016	1015	
D	976	975	
W.A	4.1%	4.1%	**4.1%**
Density (SSD)	2.484	2.488	**2.486**
Density (O.Dry)	2.386	2.390	**2.388**

10 mm RCCA			
Wt (gm)	S1	S2	**Avg**
B	2705	2705	
C	2100	2100	
A	1018	1019	
D	960	961	
W.A	6.0%	6.04%	**6.0%**
Density (SSD)	2.465	2.461	**2.463**
Density (O.Dry)	2.324	2.321	**2.323**

Fine Aggregate RCFA			
Wt (gm)	S1	S2	**Avg**
B	2395	2395	
C	2100	2100	
A	508	507	
D	480	479	
W.A	5.8%	5.8%	**5.8%**
Density (SSD)	2.385	2.392	**2.388**
Density (O.Dry)	2.254	2.259	**2.256**

Jaw Crusher

A.I.V	
S1	25.4
S2	22.7
S3	-
A.I.V	**24**
	24

TFV	
S1	174
S2	135
S3	-
TFV	**155**
	160

F.I	
S1	15%
S2	14%
S3	-
F.I	**15 %**
	15%

AN	
S1	6
S2	6
S3	-
AN	**6**
	6

14 mm RCCA			
Wt (gm)	S1	S2	Avg
B	2669	2672	
C	2069	2069	
A	1018	1021	
D	972	972	
W.A	4.7%	5.0%	**4.9%**
Density (SSD)	2.435	2.443	**2.439**
Density (O.Dry)	2.325	2.325	**2.325**

10 mm RCCA			
Wt (gm)	S1	S2	Avg
B	2697	2699	
C	2100	2100	
A	1014	1021	
D	961	971	
W.A	5.5%	5.1%	**5.3%**
Density (SSD)	2.432	2.419	**2.426**
Density (O.Dry)	2.305	2.301	**2.303**

Fine Aggregate RCFA			
Wt (gm)	S1	S2	Avg
B	2362	2362	
C	2068	2068	
A	497	498	
D	466	467	
W.A	6.65%	6.64%	**6.6%**
Density (SSD)	2.448	2.441	**2.445**
Density (O.Dry)	2.296	2.289	**2.292**

Natural Coarse Limestone and Gravel Sand

A.I.V	
S1	22
S2	20
S3	-
A.I.V	**21**
	20

TFV	
S1	151
S2	153
S3	-
TFV	**152**
	150

F.I	
S1	7
S2	7
S3	-
F.I	**7%**
	7

AN	
S1	3
S2	3
S3	-
AN	**3**
	3

14 mm			
Wt (gm)	S1	S2	**Avg**
B	2720	2721	
C	2101	2101	
A	995	997	
D	984	986	
W.A	**1.12%**	**1.12%**	**1.12%**
Density (SSD)	**2.646**	**2.645**	**2.646**
Density (O.Dry)	**2.617**	**2.615**	**2.616**

10 mm			
Wt (gm)	S1	S2	**Avg**
B	2681	2683	
C	2069	2069	
A	985	988	
D	972	975	
W.A	**1.34%**	**1.33%**	**1.33%**
Density (SSD)	**2.641**	**2.642**	**2.642**
Density (O.Dry)	**2.606**	**2.607**	**2.606**

Fine Aggregate			
Wt (gm)	S1	S2	**Avg**
B	2401.0	2402	
C	2100	2102	
A	486	484	
D	478	476	
W.A	**1.67%**	**1.68%**	**1.67%**
Density (SSD)	2.627	2.630	**2.629**
Density (O.Dry)	2.584	2.587	**2.586**

Slump Workability for tests reported in Chapter 5

Replacement Percentages	Concrete with RCCA Derived from								
	Cone			Impact			Jaw		
0%	60			60			60		
	60	55	60	60	55	60	60	55	60
20%	70			75			80		
	65	70	70	70	80	75	80	85	75
50%	75			125			75		
	70	75	75	120	125	130	70	75	80

Replacement Percentages	Concrete with RCFA Derived from								
	Cone			Impact			Jaw		
0%	60			60			60		
	60	55	60	60	55	60	60	55	60
20%	120			120			125		
	115	120	125	120	120	125	125	120	130
50%	45			85			55		
	45	45	45	90	80	85	55	55	50

Replacement Percentages	Concrete with both RCCA & RCFA Derived from								
	Cone			Impact			Jaw		
0%	60			60			60		
	60	55	60	60	55	60	60	55	60
20%	90			100			75		
	95	85	90	100	105	95	70	75	75
50%	30			30			30		
	30	25	35	30	25	30	25	30	30

Compacting factor Workability for tests reported in Chapter 5

Replacement Percentages	Concrete with RCCA Derived from					
	Cone		Impact		Jaw	
0%	0.98		0.98		0.98	
	0.980	0975	0.980	0975	0.980	0975
20%	0.96		0.97		0.98	
	0.955	0.965	0.970	0.965	0.985	0.975
50%	0.95		0.97		0.97	
	0.950	0.950	0.970	0.970	0.965	0.970

Replacement Percentages	Concrete with RCFA Derived from					
	Cone		Impact		Jaw	
0%	0.98		0.98		0.98	
	0.980	0975	0.980	0975	0.980	0975
20%	0.97		0.99		0.98	
	0.975	0.965	0.990	0.990	0.975	0.980
50%	0.92		0.97		0.93	
	0.92	0.92	0.975	0.965	0.930	0.930

Replacement Percentages	Concrete with both RCCA & RCFA Derived from					
	Cone		Impact		Jaw	
0%	0.98		0.98		0.98	
	0.980	0975	0.980	0975	0.980	0975
20%	0.93		0.98		0.96	
	0.930	0.925	0.975	0.985	0.960	0.960
50%	0.86		0.93		0.93	
	0.860	0.860	0.930	0.925	0.935	0.930

Compressive Cube Strength Mean and Standard Deviation (N/mm2) for specimens reported in Chapter 5

NA 100%

Days	Mean	STDEV
7	42.8	0.8
14	52.8	0.3
28	61.8	1.5

RCCA 20%

	Cone		Impact		Jaw	
Days	Mean	STDEV	Mean	STDEV	Mean	STDEV
7	43.2	1.9	38.1	0.4	38.5	0.5
14	51.8	2.1	49.6	1.6	45.5	0.5
28	65.5	0.5	63.0	1.7	58.7	0.6

RCCA 50%

	Cone		Impact		Jaw	
Days	Mean	STDEV	Mean	STDEV	Mean	STDEV
7	43.8	0.6	38.9	0.5	38.1	1.4
14	54.8	0.6	47.9	0.8	50.2	1.9
28	67.3	2.3	60.7	0.3	63.8	3.0

RCFA 20%

	Cone		Impact		Jaw	
Days	Mean	STDEV	Mean	STDEV	Mean	STDEV
7	38.2	0.6	38.0	0.7	33.6	1.1
14	48.3	2.5	45.9	1.1	40.5	0.6
28	60.8	0.6	59.0	1.7	53.3	2.6

RCFA 50%

	Cone		Impact		Jaw	
Days	Mean	STDEV	Mean	STDEV	Mean	STDEV
7	46.0	0.0	43.3	4.7	37.1	0.2
14	52.7	1.8	50.4	2.5	47.6	0.6
28	65.2	1.0	64.5	1.3	58.8	1.2

RCCA & RCFA 20%

	Cone		Impact		Jaw	
Days	Mean	STDEV	Mean	STDEV	Mean	STDEV
7	39.8	2.4	38.2	1.6	36.7	2.1
14	51.7	0.6	52.8	2.8	49.0	3.1
28	64.3	1.5	65.0	1.0	63.5	0.5

RCCA & RCFA 50%

	Cone		Impact		Jaw	
Days	Mean	STDEV	Mean	STDEV	Mean	STDEV
7	46.7	1.5	41.3	0.8	39.9	0.3
14	54.0	1.0	50.9	1.1	51.5	0.6
28	64.6	0.3	63.8	1.0	65.7	0.3

Tensile splitting strength Mean and Standard Deviation (N/mm^2) for specimens reported in Chapter 5

NA 100%		
Days	Mean	STDEV
7	3.52	0.06
14	3.49	0.33
28	4.30	0.26

RCCA 20%

	Cone		Impact		Jaw	
Days	Mean	STDEV	Mean	STDEV	Mean	STDEV
7	3.32	0.28	2.97	0.16	3.07	0.19
14	2.40	0.30	3.62	0.29	3.38	0.32
28	4.05	0.57	4.40	0.20	4.39	0.11

RCCA 50%

	Cone		Impact		Jaw	
Days	Mean	STDEV	Mean	STDEV	Mean	STDEV
7	3.37	0.11	2.97	0.16	3.09	0.08
14	3.55	0.06	3.64	0.15	3.39	0.21
28	4.77	0.06	4.48	0.02	4.02	0.13

RCFA 20%

	Cone		Impact		Jaw	
Days	Mean	STDEV	Mean	STDEV	Mean	STDEV
7	3.32	0.08	3.09	0.03	2.85	0.23
14	3.92	0.02	3.90	0.15	3.31	0.13
28	4.03	0.06	4.60	0.09	4.07	0.09

RCFA 50%

	Cone		Impact		Jaw	
Days	Mean	STDEV	Mean	STDEV	Mean	STDEV
7	3.22	0.10	3.13	0.09	3.13	0.09
14	3.85	0.41	3.88	0.02	3.88	0.02
28	4.47	0.36	4.21	0.09	4.21	0.09

RCCA & RCFA 20%

	Cone		Impact		Jaw	
Days	Mean	STDEV	Mean	STDEV	Mean	STDEV
7	3.1	0.3	3.1	0.1	3.1	0.1
14	3.7	0.1	3.8	0.2	3.6	0.2
28	4.0	0.2	4.2	0.3	3.8	0.1

RCCA & RCFA 50%

	Cone		Impact		Jaw	
Days	Mean	STDEV	Mean	STDEV	Mean	STDEV
7	3.6	0.0	3.4	0.1	3.1	0.2
14	4.0	0.1	3.9	0.1	3.8	0.1
28	4.6	0.1	4.7	0.2	4.9	0.0

Flexural Strength Mean (prisms) and Standard Deviation (N/mm^2) for specimens reported in Chapter

NA 100%

Days	Mean	STDEV
7	4.23	0.00
14	5.03	0.57
28	5.13	0.04

RCCA 20%

	Cone		Impact		Jaw	
Days	Mean	STDEV	Mean	STDEV	Mean	STDEV
7	4.45	0.07	4.52	0.11	4.52	0.06
14	5.04	0.00	5.52	0.47	5.37	0.21
28	5.81	0.61	6.43	0.31	6.06	0.21

RCCA 50%

	Cone		Impact		Jaw	
Days	Mean	STDEV	Mean	STDEV	Mean	STDEV
7	5.09	0.22	4.52	0.02	4.41	0.00
14	5.38	0.07	5.34	0.04	5.00	0.11
28	5.40	1.04	6.21	0.22	5.87	0.06

RCFA 20%

	Cone		Impact		Jaw	
Days	Mean	STDEV	Mean	STDEV	Mean	STDEV
7	5.17	0.12	4.52	0.23	4.13	0.40
14	6.00	0.52	5.09	0.19	4.83	0.64
28	5.68	0.24	6.28	0.32	5.94	0.36

RCFA 50%

	Cone		Impact		Jaw	
Days	Mean	STDEV	Mean	STDEV	Mean	STDEV
7	5.19	0.06	4.79	0.19	4.41	0.17
14	5.11	3.62	5.42	0.02	5.25	0.21
28	6.74	0.59	6.61	0.12	6.48	0.30

RCCA & RCFA 20%

	Cone		Impact		Jaw	
Days	Mean	STDEV	Mean	STDEV	Mean	STDEV
7	5.4	0.3	4.8	0.3	4.5	0.0
14	6.2	0.6	5.3	0.4	5.5	0.0
28	5.9	0.4	7.1	0.6	6.0	0.1

RCCA & RCFA 50%

	Cone		Impact		Jaw	
Days	Mean	STDEV	Mean	STDEV	Mean	STDEV
7	5.3	0.2	4.9	0.1	4.9	0.1
14	6.0	0.3	5.6	0.4	5.8	0.1
28	6.9	0.6	5.8	0.3	6.3	0.2

S. S. Density mean (prisms) and Standard Deviation (Kg/m^3) for specimens reported in Chapter 5

NA 100%

Days	Mean	STDEV
7	2407	2.9
14	2412	2.9
28	2418	2.9

RCCA 20%

	Cone		Impact		Jaw	
Days	Mean	STDEV	Mean	STDEV	Mean	STDEV
7	2403	2.9	2402	2.9	2380	5.0
14	2400	5.0	2402	7.6	2385	8.7
28	2412	2.9	2405	5.0	2390	5.0

RCCA 50%

	Cone		Impact		Jaw	
Days	Mean	STDEV	Mean	STDEV	Mean	STDEV
7	2382	2.9	2372	2.9	2383	2.9
14	2385	0.0	2373	2.9	2390	18.3
28	2388	5.8	2382	2.5	2393	8.7

RCFA 20%

	Cone		Impact		Jaw	
Days	Mean	STDEV	Mean	STDEV	Mean	STDEV
7	2417	7.6	2395	0.0	2393	10.4
14	2418	7.6	2392	2.9	2388	2.9
28	2423	2.9	2395	5.0	2400	0.0

RCFA 50%

	Cone		Impact		Jaw	
Days	Mean	STDEV	Mean	STDEV	Mean	STDEV
7	2407	2.9	2385	5.0	2382	2.9
14	2417	24.7	2392	2.9	2383	2.9
28	2405	5.0	2392	7.6	2385	0.0

RCCA & RCFA 20%

	Cone		Impact		Jaw	
Days	Mean	STDEV	Mean	STDEV	Mean	STDEV
7	2395	5.0	2377	7.6	2385	5.0
14	2408	2.9	2397	2.9	2397	12.6
28	2412	5.8	2395	8.7	2400	5.0

RCCA & RCFA 50%

	Cone		Impact		Jaw	
Days	Mean	STDEV	Mean	STDEV	Mean	STDEV
7	2378	2.9	2372	2.9	2362	5.8
14	2375	5.0	2372	2.9	2370	8.7
28	2362	5.8	2370	5.0	2365	0.0

Cone Tensile Splitting Strength

Control Mix NCA 100 %

Day Tested	Test No.	Max. Load (N)	Strength N/mm^2
3	1	246.00	3.479
3	2	252.00	3.564
7	1	263.00	3.719
7	2	230.00	3.253
28	1	317	4.483
28	2	291	4.115

Mix # 1 RCCA 20 %

Day Tested	Test No.	Max. Load (N)	Strength N/mm^2
3	1	249	3.521
3	2	221	3.125
7	1	155	2.192
7	2	185	2.616
28	1	258	3.648
28	2	315	4.455

Mix # 2 RCCA 50 %

Day Tested	Test No.	Max. Load (N)	Strength N/mm^2
3	1	244	3.451
3	2	233	3.295
7	1	254	3.592
7	2	248	3.507
28	1	340	4.808
28	2	334	4.723

Mix # 3 RCFA 20 %

Day Tested	Test No.	Max. Load (N)	Strength N/mm^2
3	1	239	3.380
3	2	231	3.267
7	1	276	3.903
7	2	278	3.931
28	1	282	3.988
28	2	288	4.073

Mix # 4 RCFA 50 %

Day Tested	Test No.	Max. Load (N)	Strength N/mm^2
3	1	223	3.154
3	2	233	3.295
7	1	252	3.564
7	2	293	4.143
28	1	334	4.723
28	2	298	4.214

Mix # 5 RCCA 20 % & RCFA 20 %

Day Tested	Test No.	Max. Load (N)	Strength N/mm^2
3	1	204	2.885
3	2	235	3.323
7	1	265	3.747
7	2	257	3.634
28	1	303	4.285
28	2	323	4.568

Mix # 6 RCCA 50 % & RCFA 50 %

Day Tested	Test No.	Max. Load (N)	Strength N/mm^2
3	1	253	3.578
3	2	250	3.535
7	1	275	3.889
7	2	288	4.073
28	1	325	4.596
28	2	319	4.511

Cone Flexural Strength

Control NCA 100 %

Day	Test No.	Max. Load (KN)	Strength N/mm2
3	1	14.1	4.23
7	1	15.4	4.62
7	2	18.1	5.43
28	1	17.029	5.1087
28	2	17.195	5.1585

Mix # 1 RCCA 20 %

Day	Test No.	Max. Load (N)	Strength N/mm2
3	1	15.00	4.500
3	2	14.67	4.401
7	1	16.80	5.040
7	2	16.80	5.040
28	1	20.795	6.239
28	2	17.913	5.374

Mix # 2 RCCA 50%

Day	Test No.	Max. Load (N)	Strength N/mm2
3	1	16.45	4.936
3	2	17.48	5.244
7	1	18.12	5.436

7	2	17.77	5.332
28	1	15.544	4.663
28	2	20.462	6.139

Mix #3 RCFA 50%

Day	Test No.	Max. Load (N)	Strength N/mm2
3	1	16.77	5.032
3	2	17.57	5.272
3	3	17.574	5.272
3	4	17.00	5.099
7	1	21.58	6.473
7	2	19.54	5.862
7	3	20.25	6.075
7	4	18.598	5.579
28	1	18.368	5.510
28	2	19.52	5.856

Mix # 4 RCFA 50%

Day	Test No.	Max. Load (N)	Strength N/mm2
3	1	17.43	5.228
3	2	17.15	5.144
7	1	0.00	0.000
7	2	17.04	5.113
28	1	23.857	7.157
28	2	21.09	6.327

Mix # 5 RCCA 20% RCFA 20%

Day	Test No.	Max. Load (N)	Strength N/mm2
3	1	17.34	5.201
3	2	18.56	5.567
7	1	19.17	5.751
7	2	21.94	6.581
28	1	18.65	5.595
28	2	19.674	5.902
28	3	21.1	6.330

Mix # 6 RCCA 50% RCFA 50 %

Day	Test No.	Max. Load (N)	Strength N/mm2
3	1	17.24	5.171
3	2	18.03	5.409
7	1	20.76	6.227

7	2	19.25	5.774
28	1	24.384	7.315
28	2	21.67	6.501

Cone Compressive Strength

Mix # 0 RCCA :0% (Control Mix)

Test No.	Day Tested	Max. Load (KN)	Comp. S. (N/mm^2)
1	3	422	42.0
2	3	433	43.5
3	3	430	43.0
1	7	528	53.0
2	7	528	53.0
3	7	524	52.5
1	28	605	60.5
2	28	635	63.5
3	28	615	61.5

Mix # 1 RCCA :20%

Test No.	Day Tested	Max. Load (KN)	Comp. S. (N/mm^2)
1	3	412	41.0
2	3	447	44.5
3	3	438	44.0
1	7	526	52.5
2	7	535	53.5
3	7	494	49.5
1	28	650	65.0
2	28	655	65.5
3	28	660	66.0
4	28	660	66.0

Mix # 2 RCCA :50%

Test No.	Day Tested	Max. Load (KN)	Comp. S. (N/mm^2)
1	3	436	43.5
2	3	434	43.5
3	3	444	44.5
1	7	545	54.5
2	7	554	55.5
3	7	546	54.5
1	28	660	66.0
2	28	660	66.0
3	28	700	70.0

Mix # 3 RFCA :20%

Test No.	Day Tested	Max. Load (KN)	Comp. S. (N/mm^2)
1	3	386	38.5
2	3	373	37.5
3	3	385	38.5
1	7	494	49.5
2	7	456	45.5
3	7	498	50.0
1	28	615	61.5
2	28	605	60.5
3	28	605	60.5

Mix # 4 RFCA :50%

Test No.	Day Tested	Max. Load (KN)	Comp. S. (N/mm^2)
1	3	462	46.0
2	3	462	46.0
3	3	462	46.0
1	7	544	54.5
2	7	526	52.5
3	7	508	51.0
1	28	655	65.5
2	28	660	66.0
3	28	640	64.0

Mix # 5 RFCA :20 % RCCA: 20%

Test No.	Day Tested	Max. Load (KN)	Comp. S. (N/mm^2)
1	3	388	39.0
2	3	424	42.5
3	3	37.9	38.0
1	7	520	52.0
2	7	510	51.0
3	7	518	52.0
1	28	660	66.0
2	28	640	64.0
3	28	630	63.0

Mix # 6 RFCA :50 % RCCA: 50%

Test No.	Day Tested	Max. Load (KN)	Comp. S. (N/mm^2)
1	3	478	48.0
2	3	451	45.0
3	3	468	47.0
1	7	550	55.0
2	7	532	53.0
3	7	538	54.0
1	28	645	64.5
2	28	650	65.0
3	28	645	64.5

4	28	645	64.5

Impact Flexural Strength

Controle NCA100 %

Day	Test No.	Max. Load (KN)	Strength N/mm2
3	1	14.1	4.23
7	1	15.4	4.62
7	2	18.1	5.43
28	1	17.029	5.1087
28	2	17.195	5.1585

Mix # 1 RCCA 20 %

Day	Test No.	Max. Load (N)	Strength N/mm2
3	1	15.30	4.590
3	2	14.80	4.440
7	1	17.30	5.190
7	2	19.50	5.850
28	1	20.6	6.180
28	2	21.1	6.330
28	3	22.6	6.780

Mix # 2 RCCA 50%

Day	Test No.	Max. Load (N)	Strength N/mm2
3	1	15.00	4.500
3	2	15.10	4.530
7	1	17.70	5.310
7	2	17.90	5.370
28	1	20.9	6.270
28	2	21.3	6.390
28	3	19.9	5.970

Mix #3 RCFA 20%

Day	Test No.	Max. Load (N)	Strength N/mm2
3	1	15.60	4.680
3	2	14.50	4.350
7	1	16.50	4.950
7	2	17.4	5.220
28	1	21.5	6.450

28	2	19.7	5.910
28	3	21.6	6.480

Mix # 4 RCFA 50%

Day	Test No.	Max. Load (N)	Strength N/mm2
3	1	16.40	4.920
3	2	15.50	4.650
7	1	18.10	5.430
7	2	18.00	5.400
28	1	21.8	6.540
28	2	22.5	6.750
28	3	21.8	6.540

Mix # 5 RCCA 20% RCFA 20%

Day	Test No.	Max. Load (N)	Strength N/mm2
3	1	15.50	4.650
3	2	16.80	5.040
7	1	16.60	4.980
7	2	18.60	5.580
28	1	23.40	7.020
28	2	22.00	6.600
28	3	25.80	7.740

Mix # 6 RCCA 50% RCFA 50 %

Day	Test No.	Max. Load (N)	Strength N/mm2
3	1	16.60	4.980
3	2	16.20	4.860
7	1	19.60	5.880
7	2	17.80	5.340
28	1	18.30	5.490
28	2	19.20	5.760
28	3	20.40	6.120

Impact Tensile Splitting Strength

Control Mix NCA 100 %

Day Tested	Test No.	Max. Load (N)	Strength N/mm^2
3	1	246.00	3.479
3	2	252.00	3.564
7	1	263.00	3.719
7	2	230.00	3.253
28	1	317	4.483
28	2	291	4.115

Mix # 1 RCCA 20 %

Day Tested	Test No.	Max. Load (N)	Strength N/mm^2
3	1	197	2.786
3	2	215	3.040
3	3	218	3.083
7	1	246	3.479
7	2	279	3.945
7	3	242	3.422
28	1	318	4.497
28	2	321	4.539
28	3	295	4.172

Mix # 2 RCCA 50 %

Day Tested	Test No.	Max. Load (N)	Strength N/mm^2
3	1	198	2.800
3	2	212	2.998
3	3	220	3.111
7	1	246	3.479
7	2	261	3.691
7	3	266	3.762
28	1	317	4.483
28	2	315	4.455
28	3	318	4.497

Mix # 3 RCFA 20 %

Day Tested	Test No.	Max. Load (N)	Strength N/mm^2
3	1	216	3.055
3	2	220	3.111
3	3	220	3.111
7	1	264	3.733
7	2	279	3.945
7	3	285	4.030
28	1	332	4.695
28	2	324	4.582
28	3	319	4.511

Mix # 4 RCFA 50 %

Day Tested	Test No.	Max. Load (N)	Strength N/mm^2
3	1	221	3.125
3	2	215	3.040
3	3	228	3.224
7	1	274	3.875
7	2	276	3.903
7	3	273	3.861
28	1	301	4.257
28	2	302	4.271
28	3	291	4.115

Mix # 5 RCCA 20 % RCFA 20

Day Tested	Test No.	Max. Load (N)	Strength N/mm^2
3	1	220	3.111
3	2	214	3.026
3	3	230	3.253
7	1	262	3.705
7	2	287	4.059
7	3	260	3.677
28	1	331	4.681
28	2	304	4.299
28	3	339	4.794

Mix # 6 RCCA 50 % RCFA 50 %

Day Tested	Test No.	Max. Load (N)	Strength N/mm^2
3	1	250	3.535
3	2	234	3.309
3	3	241	3.408
7	1	288	4.073
7	2	271	3.832
7	3	275	3.889
28	1	343	4.851
28	2	320	4.525
28	3	339	4.794

Impact Compressive Strength

Mix # 0 (Control Mix)

Test No.	Day Tested	Max. Load (KN)	Comp. S. (N/mm^2)
1	3	422	42.0
2	3	433	43.5
3	3	430	43.0
1	7	528	53.0
2	7	528	53.0

3	7	524	52.5
1	28	605	60.5
2	28	635	63.5
3	28	615	61.5

Mix # 1 RCCA :20%

Test No.	Day Tested	Max. Load (KN)	Comp. S. (N/mm^2)
1	3	383	38.3
2	3	384	38.4
3	3	376	37.6
1	7	485	48.5
2	7	514	51.4
3	7	488	48.8
1	28	640	64.0
2	28	610	61.0
3	28	640	64.0

Mix # 2 RCCA :50%

Test No.	Day Tested	Max. Load (KN)	Comp. S. (N/mm^2)
1	3	394	39.4
2	3	387	38.7
3	3	385	38.5
1	7	486	48.6
2	7	481	48.1
3	7	470	47.0
1	28	610	61.0
2	28	605	60.5
3	28	605	60.5

Mix # 3 RFCA :20%

Test No.	Day Tested	Max. Load (KN)	Comp. S. (N/mm^2)
1	3	382	38.2
2	3	372	37.2
3	3	385	38.5
1	7	470	47.0
2	7	448	44.8
3	7	460	46.0
1	28	580	58.0
2	28	610	61.0
3	28	580	58.0

Mix # 4 RFCA :50%

Test No.	Day Tested	Max. Load (KN)	Comp. S. (N/mm^2)
1	3	379	37.9
2	3	360	46.0
3	3	371	46.0
1	7	477	47.7

2	7	482	52.5
3	7	502	51.0
1	28	635	63.5
2	28	620	66.0
3	28	600	64.0

Mix # 5 RFCA : 20 % RCCA: 20%

Test No.	Day Tested	Max. Load (KN)	Comp. S. (N/mm^2)
1	3	368	36.8
2	3	400	40.0
3	3	378	37.8
1	7	555	55.5
2	7	530	53.0
3	7	500	50.0
1	28	640	64.0
2	28	650	65.0
3	28	660	66.0

Mix # 6 RFCA : 50 % RCCA: 50%

Test No.	Day Tested	Max. Load (KN)	Comp. S. (N/mm^2)
1	3	408	40.8
2	3	408	40.8
3	3	422	42.2
1	7	517	51.7
2	7	497	49.7
3	7	514	51.4
1	28	630	63.0
2	28	635	63.5
3	28	650	65.0

Jaw Flexural Strength

Control NCA 100%

Day	Test No.	Max. Load (KN)	Strength N/mm2
3	1	14.1	4.23
7	1	15.4	4.62
7	2	18.1	5.43
28	1	17.029	5.1087
28	2	17.195	5.1585

Mix # 1 RCCA 20 %

Day	Test No.	Max. Load (N)	Strength N/mm2
3	1	15.20	4.560
3	2	14.90	4.470
7	1	18.40	5.520
7	2	17.40	5.220

28	1	20.7	6.210
28	2	20.5	6.150
28	3	19.4	5.820

Mix # 2 RCCA 50%

Day	Test No.	Max. Load (N)	Strength N/mm2
3	1	14.70	4.410
3	2	14.70	4.410
7	1	16.90	5.070
7	2	16.40	4.920
28	1	19.4	5.820
28	2	19.7	5.910

Mix #3 RCFA 50%

Day	Test No.	Max. Load (N)	Strength N/mm2
3	1	12.80	3.840
3	2	14.70	4.410
7	1	14.60	4.380
7	2	17.6	5.280
28	1	21.1	6.330
28	2	18.7	5.610
28	3	19.6	5.880

Mix # 4 RCFA 50%

Day	Test No.	Max. Load (N)	Strength N/mm2
3	1	14.30	4.290
3	2	15.10	4.530
7	1	17.00	5.100
7	2	18.00	5.400
28	1	20.5	6.150
28	2	22.4	6.720
28	3	21.9	6.570

Mix # 5 RCCA 20% RCFA 20%

Day	Test No.	Max. Load (N)	Strength N/mm2
3	1	15.00	4.500
3	2	15.20	4.560
7	1	18.40	5.520
7	2	18.30	5.490
28	1	19.6	5.880
28	2	20.4	6.120
28	3	20	6.000

Mix # 6 RCCA 50% RCFA 50 %

Day	Test No.	Max. Load (N)	Strength N/mm2
3	1	16.20	4.860

3	2	16.50	4.950
7	1	19.30	5.790
7	2	19.60	5.880
28	1	20.30	6.090
28	2	20.80	6.240
28	3	21.40	6.420

Jaw Tensile Splitting Strength

Control Mix NCA 100 %

Day Tested	Test No.	Max. Load (N)	Strength N/mm^2
3	1	246.00	3.479
3	2	252.00	3.564
7	1	263.00	3.719
7	2	230.00	3.253
28	1	317	4.483
28	2	291	4.115

Mix # 1 RCCA 20 %

Day Tested	Test No.	Max. Load (N)	Strength N/mm^2
3	1	202	2.857
3	2	220	3.111
3	3	229	3.238
7	1	264	3.733
7	2	221	3.125
7	3	232	3.281
28	1	304	4.299
28	2	309	4.370
28	3	319	4.511

Mix # 2 RCCA 50 %

Day Tested	Test No.	Max. Load (N)	Strength N/mm^2
3	1	214	3.026
3	2	217	3.069
3	3	225	3.182
7	1	225	3.182
7	2	240	3.394
7	3	254	3.592
28	1	284	4.016
28	2	275	3.889
28	3	294	4.158

Mix # 3 RCFA 20 %

Day Tested	Test No.	Max. Load (N)	Strength N/mm^2

3	1	218	3.083
3	2	201	2.842
3	3	186	2.630
7	1	245	3.465
7	2	228	3.224
7	3	230	3.253
28	1	295	4.172
28	2	285	4.030
28	3	283	4.002

Mix # 4 RCFA 50 %

Day Tested	Test No.	Max. Load (N)	Strength N/mm^2
3	1	221	3.125
3	2	215	3.040
3	3	228	3.224
7	1	274	3.875
7	2	276	3.903
7	3	273	3.861
28	1	301	4.257
28	2	302	4.271
28	3	291	4.115

Mix # 5 RCCA 20 % RCFA 20 %

Day Tested	Test No.	Max. Load (N)	Strength N/mm^2
3	1	213	3.012
3	2	224	3.168
3	3	220	3.111
7	1	264	3.733
7	2	262	3.705
7	3	234	3.309
28	1	302	4.271
28	2	315	4.455
28	3	316	4.469

Mix # 6 RCCA 50 % RCFA 50 %

Day Tested	Test No.	Max. Load (N)	Strength N/mm^2
3	1	215	3.040
3	2	214	3.026
3	3	234	3.309
7	1	263	3.719
7	2	277	3.917
7	3	257	3.634
28	1	341	4.822
28	2	346	4.893
28	3	346	4.893

Jaw Compressive Strength

Mix # 0 (Control Mix)

Test No.	Day Tested	Max. Load (KN)	Comp. S. (N/mm^2)
1	3	422	42.0
2	3	433	43.5
3	3	430	43.0
1	7	528	53.0
2	7	528	53.0
3	7	524	52.5
1	28	605	60.5
2	28	635	63.5
3	28	615	61.5

Mix # 1 RCCA 20%

Test No.	Day Tested	Max. Load (KN)	Comp. S. (N/mm^2)
1	3	390	39.0
2	3	385	38.5
3	3	380	38.0
1	7	449	44.9
2	7	458	45.8
3	7	458	45.8
1	28	590	59.0
2	28	590	59.0
3	28	580	58.0

Mix # 2 RCCA 50%

Test No.	Day Tested	Max. Load (KN)	Comp. S. (N/mm^2)
1	3	367	36.7
2	3	392	39.2
3	3	372	37.2
4	3	394	39.4
1	7	486	48.6
2	7	515	51.5
3	7	522	52.2
4	7	486	48.6
1	28	610	61.0
2	28	630	63.0
3	28	630	63.0
4	28	680	68.0

Mix # 3 RFCA 20%

Test No.	Day Tested	Max. Load (KN)	Comp. S. (N/mm^2)
1	3	346	34.6
2	3	336	33.6

3	3	325	32.5
1	7	400	40.0
2	7	402	40.2
3	7	412	41.2
1	28	540	54.0
2	28	555	55.5
3	28	505	50.5

Mix # 4 RFCA 50%

Test No.	Day Tested	Max. Load (KN)	Comp. S. (N/mm^2)
1	3	370	37.0
2	3	373	37.3
3	3	370	37.0
1	7	470	47.0
2	7	480	48.0
3	7	479	47.9
1	28	595	59.5
2	28	595	59.5
3	28	575	57.5

Mix # 5 RFCA 20 %RCCA 20%

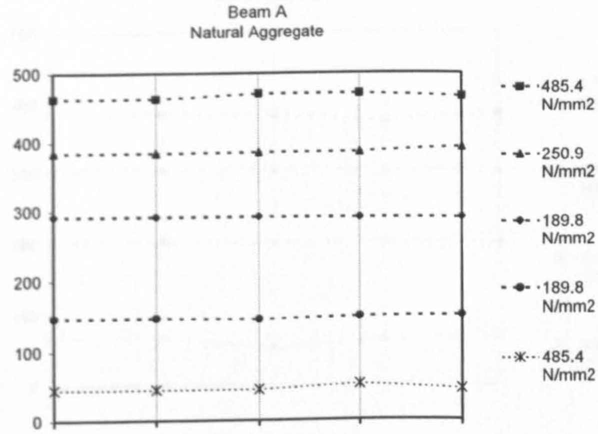
Test No.	Day Tested	Max. Load (KN)	Comp. S. (N/mm^2)
1	3	350	35.0
2	3	362	36.2
3	3	390	39.0
1	7	522	52.2
2	7	489	48.9
3	7	460	46.0
1	28	630	63.0
2	28	635	63.5
3	28	640	64.0

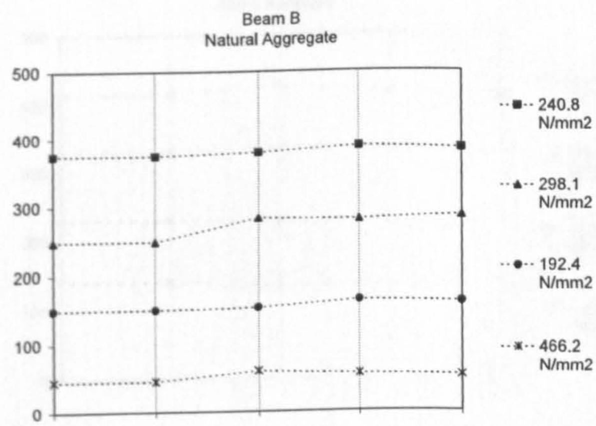
Mix # 6 RFCA 50 %RCCA 50%

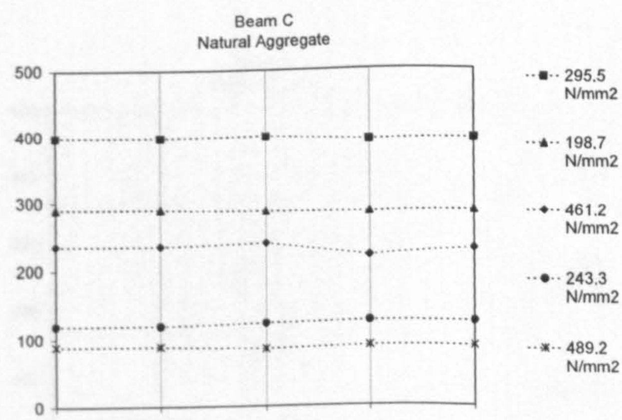
Test No.	Day Tested	Max. Load (KN)	Comp. S. (N/mm^2)
1	3	396	39.6
2	3	402	40.2
3	3	398	39.8
1	7	522	52.2
2	7	512	51.2
3	7	510	51.0
1	28	660	66.0
2	28	655	65.5
3	28	655	65.5

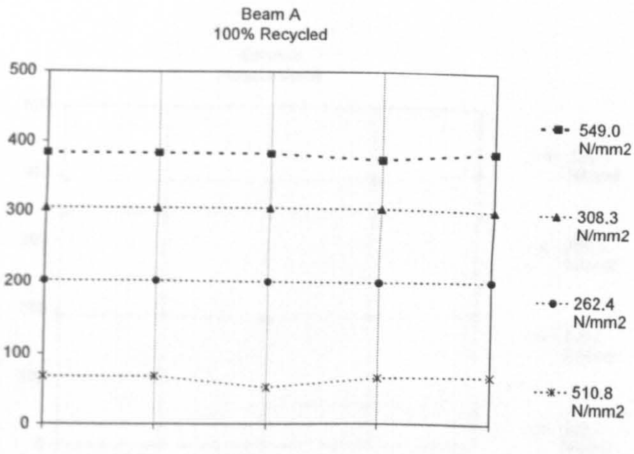
Appendix 3

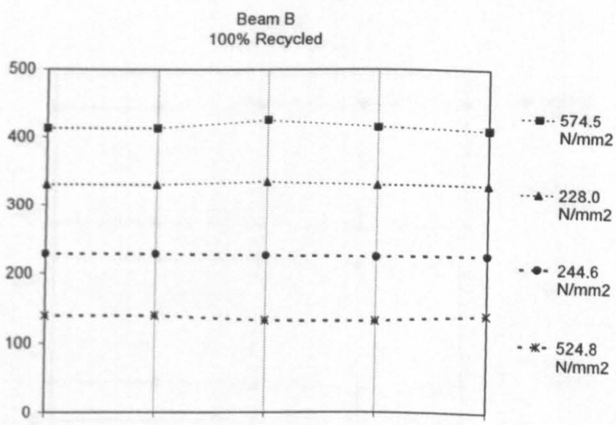
1. Tension versus elongation responses of standard concrete and reinforcement bar
2. Crack Distances (mm) for Control Concrete Prisms Reinforcement Bar (B1)
3. Tension versus elongation responses of standard concrete and pretensioning wire
4. Crack Distances (cm) for Control Concrete Prisms Pretensioning Wire (B1)
5. Force on the reinforcement Bar (P) vs. Tension force at the concrete prism-beam (Ft)
6. Force on the prestress wire (P) vs. Tension force at the prism-beam (Ft)

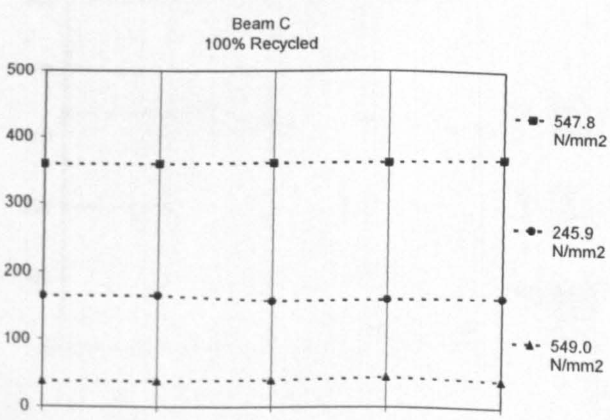

Beam A
Natural Aggregate
500
400
300
200
100
0
485.4 N/mm2
250.9 N/mm2
189.8 N/mm2
189.8 N/mm2
485.4 N/mm2

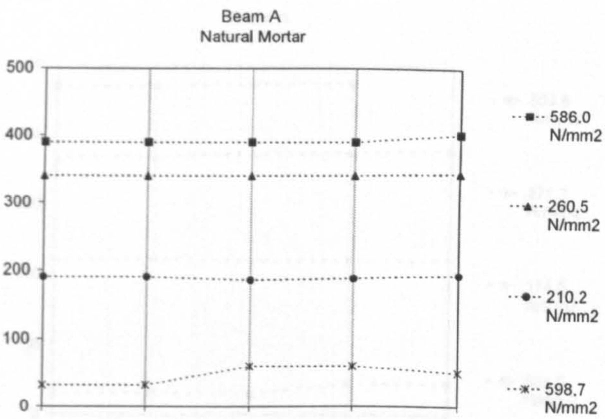

Beam B
Natural Aggregate
500
400
300
200
100
0
240.8 N/mm2
298.1 N/mm2
192.4 N/mm2
466.2 N/mm2

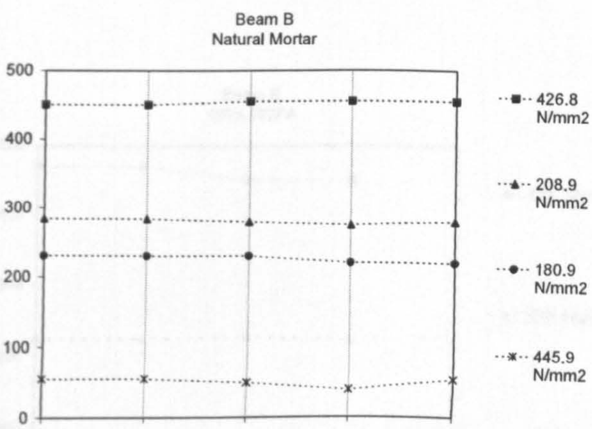

Beam C
Natural Aggregate
500
400
300
200
100
0
295.5 N/mm2
198.7 N/mm2
461.2 N/mm2
243.3 N/mm2
489.2 N/mm2

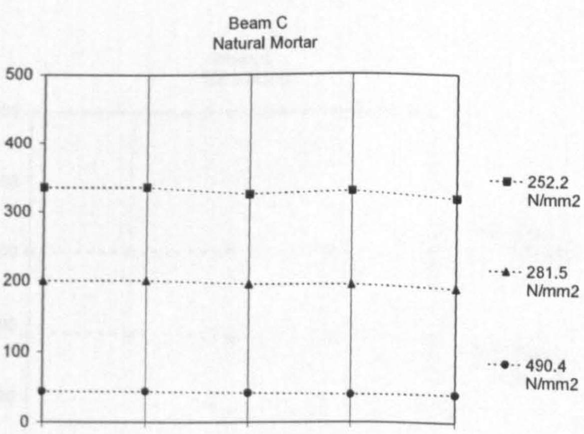

Beam A
100% Recycled
500
400
300
200
100
0
549.0 N/mm2
308.3 N/mm2
262.4 N/mm2
510.8 N/mm2

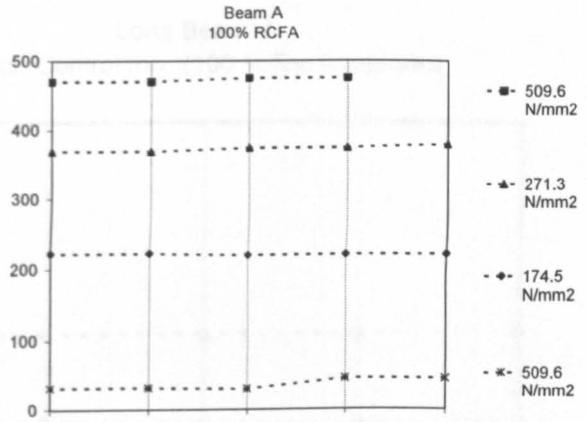

Beam B
100% Recycled
500
400
300
200
100
0
574.5 N/mm2
228.0 N/mm2
244.6 N/mm2
524.8 N/mm2

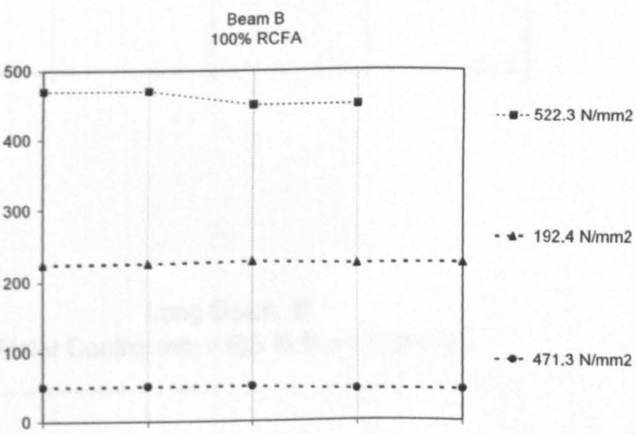

Beam C
100% Recycled
500
400
300
200
100
0
547.8 N/mm2
245.9 N/mm2
549.0 N/mm2

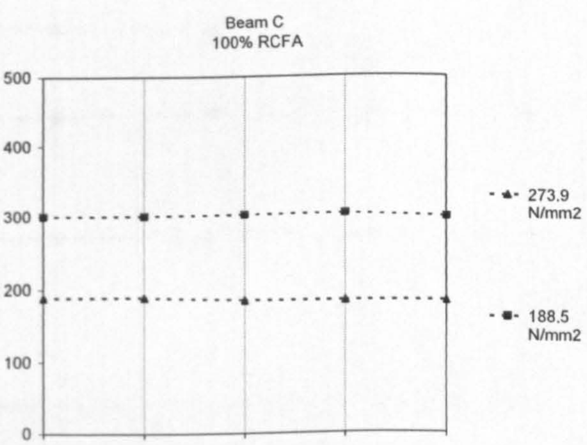

Beam A
Natural Mortar
500
400
300
200
100
0
586.0 N/mm2
260.5 N/mm2
210.2 N/mm2
598.7 N/mm2


Beam B
Natural Mortar
500
400
300
200
100
0
426.8 N/mm2
208.9 N/mm2
180.9 N/mm2
445.9 N/mm2

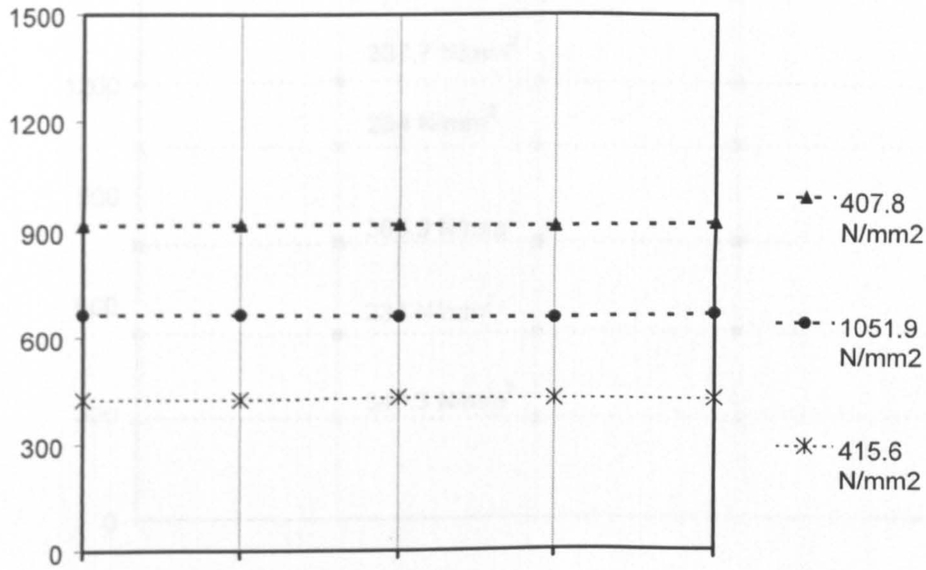

Beam C
Natural Mortar
500
400
300
200
100
0
252.2 N/mm2
281.5 N/mm2
490.4 N/mm2


Beam A
100% RCFA
500
400
300
200
100
0
509.6 N/mm2
271.3 N/mm2
174.5 N/mm2
509.6 N/mm2

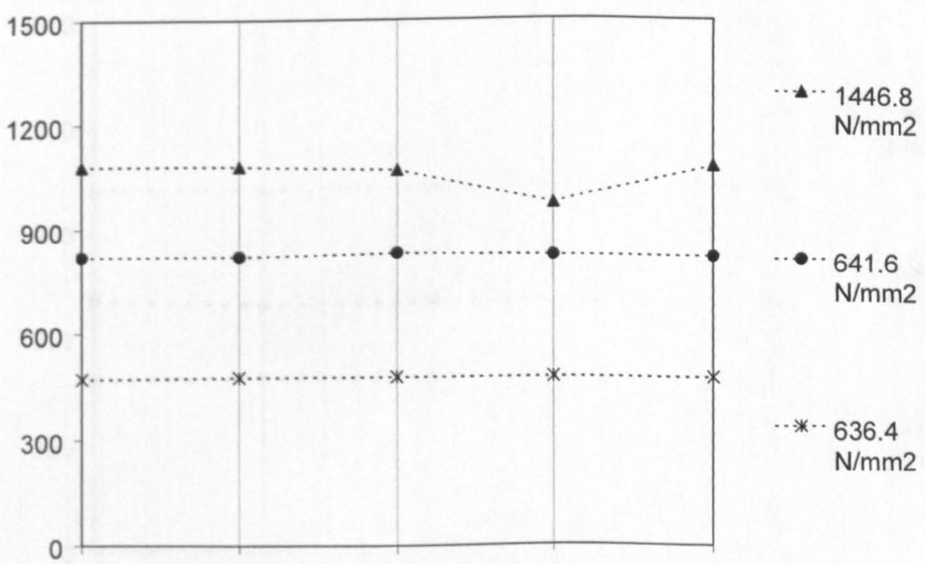

Beam B
100% RCFA
500
400
300
200
100
0
522.3 N/mm2
192.4 N/mm2
471.3 N/mm2

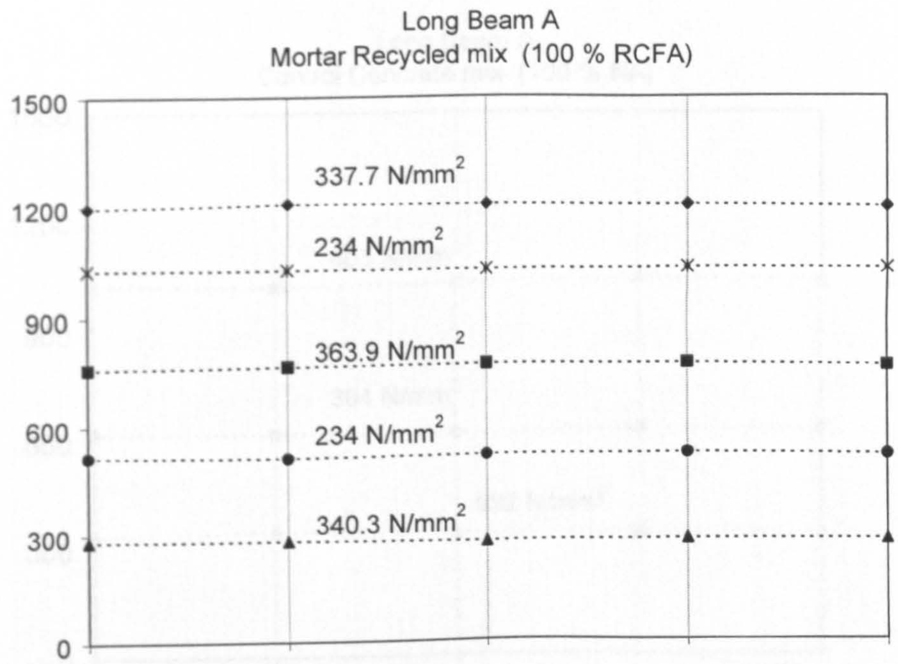

Beam C
100% RCFA
500
400
300
200
100
0
273.9 N/mm2
188.5 N/mm2

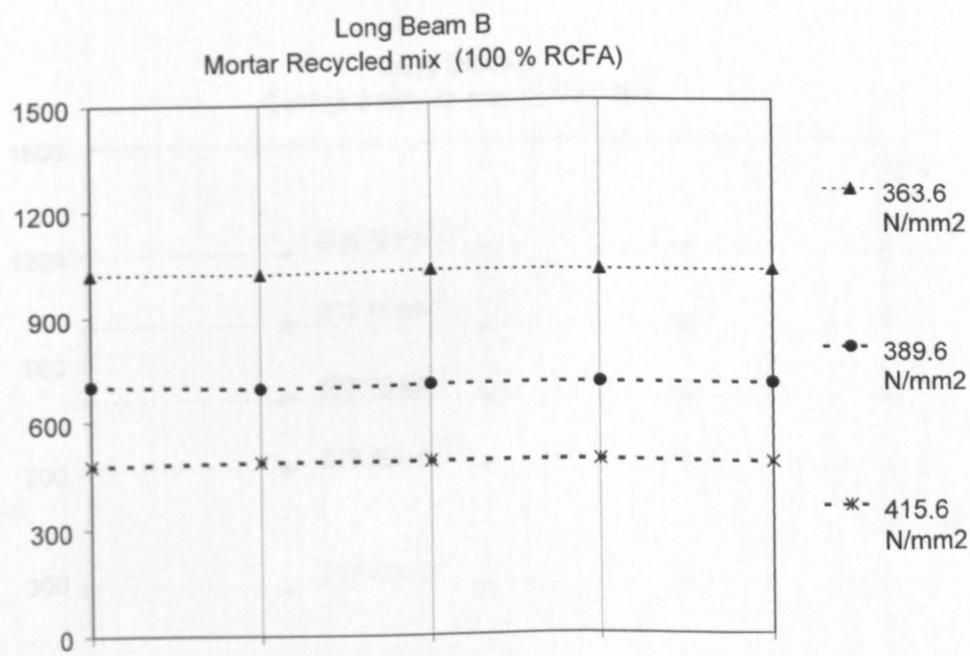
Long Beam A
Mortar Control mix (100 % fine limestone)

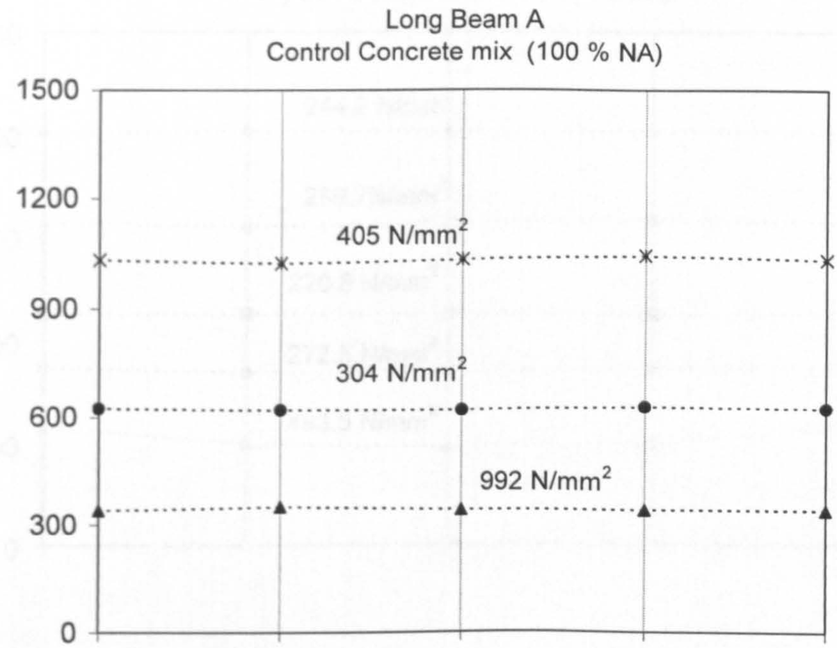


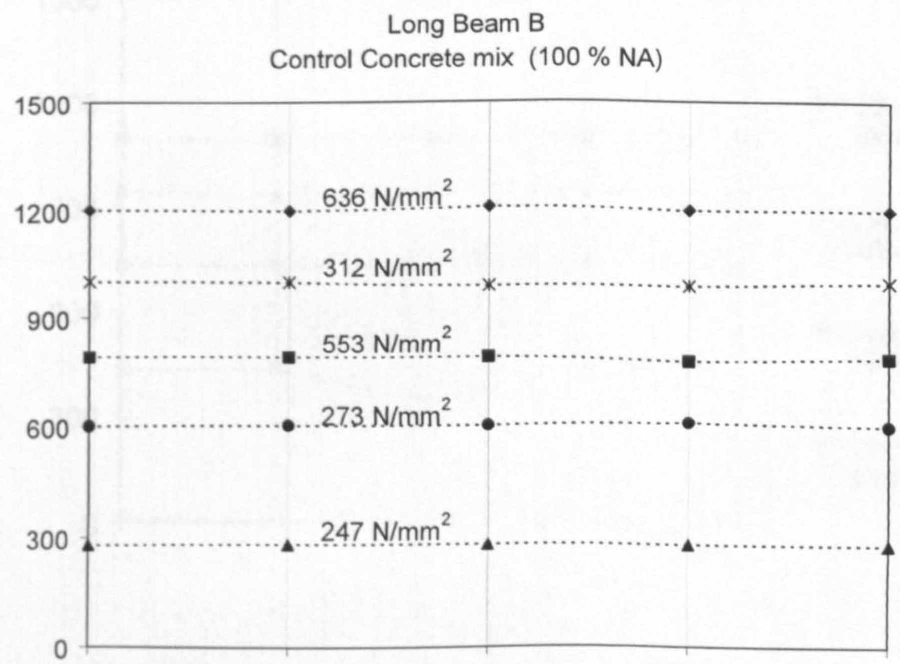
Long Beam B
Mortar Control mix (100 % fine limestone)

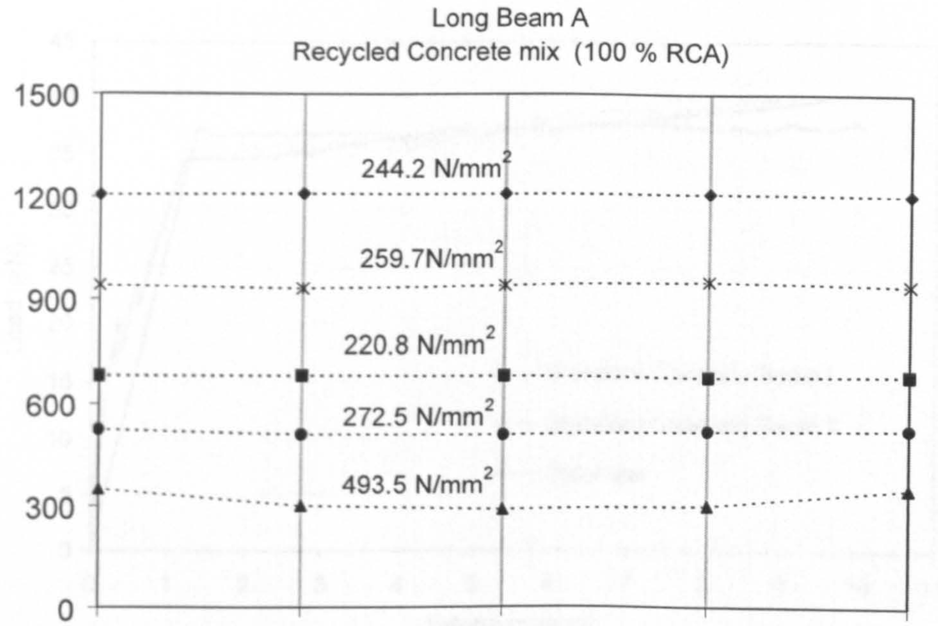



Long Beam A
Mortar Recycled mix (100 % RCFA)
1500
1200
900
600
300
0
337.7 N/mm2
234 N/mm2
363.9 N/mm2
234 N/mm2
340.3 N/mm2

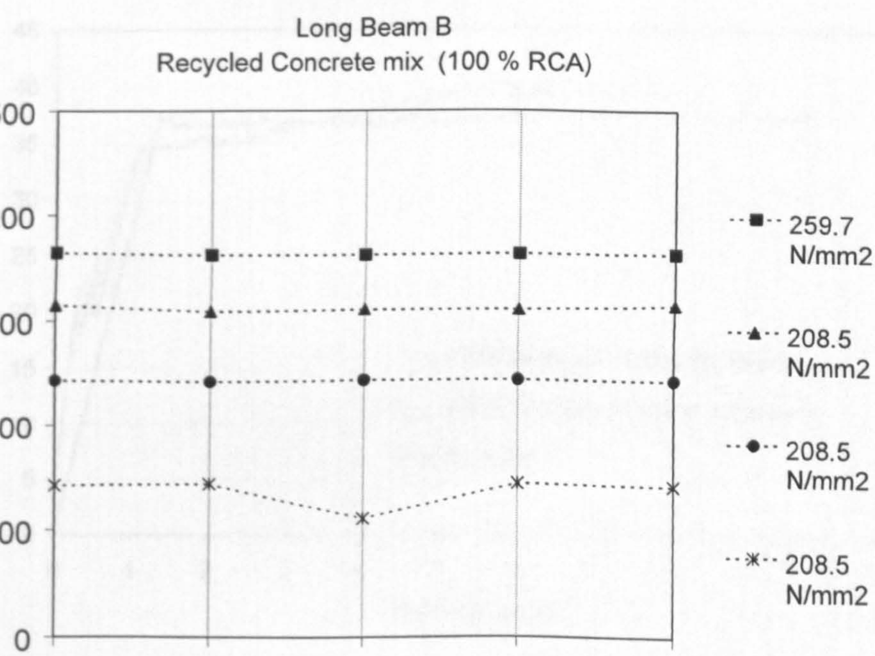

Long Beam B
Mortar Recycled mix (100 % RCFA)
1500
1200
900
600
300
0
363.6 N/mm2
389.6 N/mm2
415.6 N/mm2


Long Beam A
Control Concrete mix (100 % NA)
1500
1200
900
600
300
0
405 N/mm²
304 N/mm²
992 N/mm²

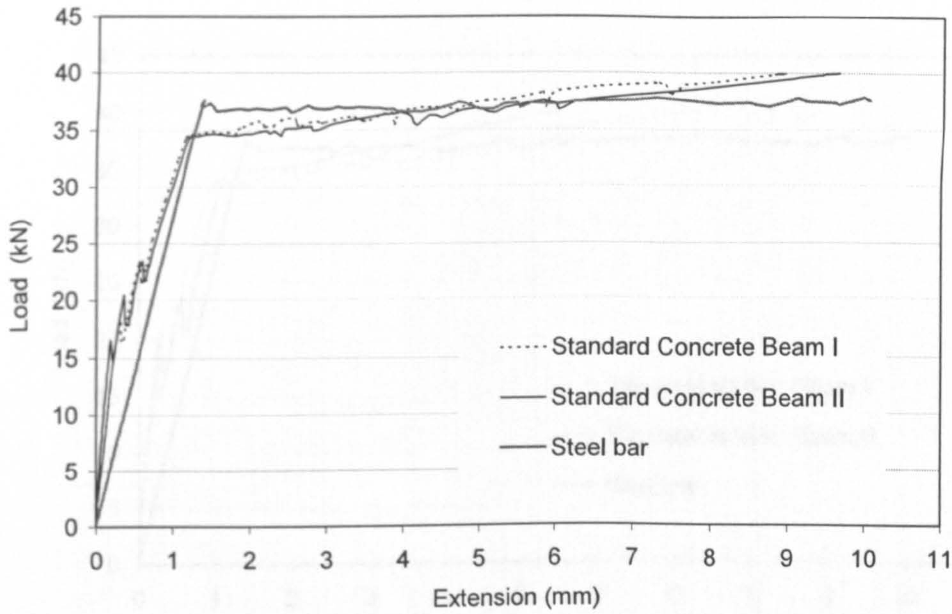

Long Beam B
Control Concrete mix (100 % NA)
1500
1200
900
600
300
0
636 N/mm²
312 N/mm²
553 N/mm²
273 N/mm²
247 N/mm²



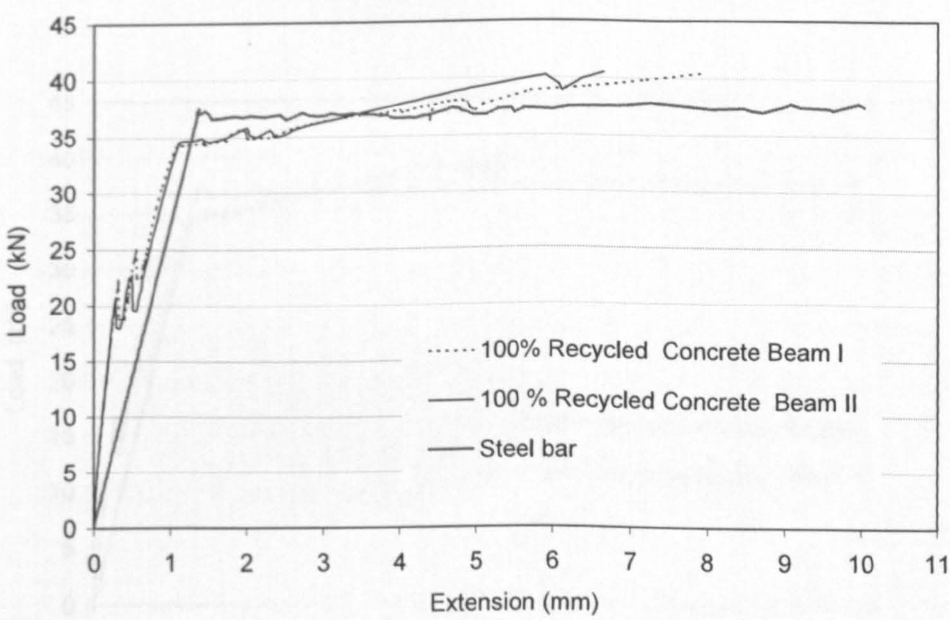
Long Beam A
Recycled Concrete mix (100 % RCA)
1500
1200
900
600
300
0
244.2 N/mm2
259.7N/mm2
220.8 N/mm2
272.5 N/mm2
493.5 N/mm2



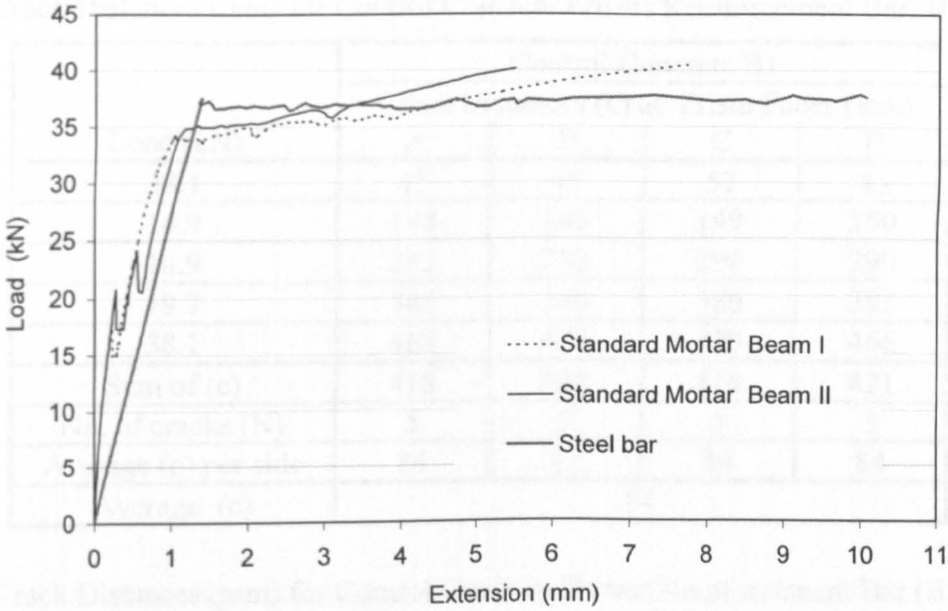
Long Beam B
Recycled Concrete mix (100 % RCA)
1500
1200
900
600
300
0
259.7 N/mm2
208.5 N/mm2
208.5 N/mm2
208.5 N/mm2



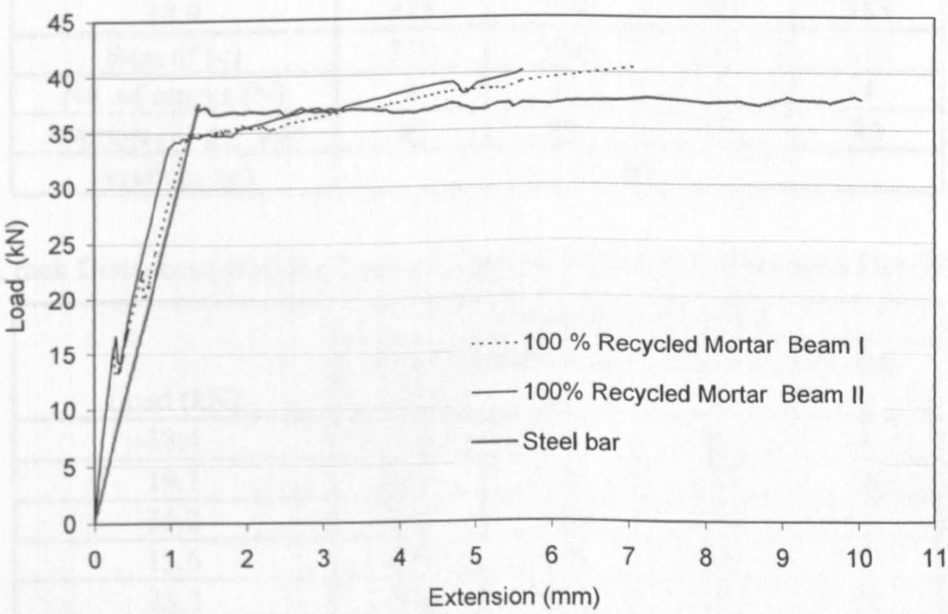
Tension versus elongation responses of standard concrete and reinforcement bar



Tension versus elongation responses of 100% RCA concrete and reinforcement bar



Tension versus elongation responses of standard mortar and reinforcement bar



Tension versus elongation responses of 100% RCFA mortar and reinforcement bar

Crack Distances (mm) for Control Concrete Prisms Reinforcement Bar (B1)

Load (kN)	Control Concrete B1 Crack Distances (c) at Prism Faces (mm) A	B	C	D
38.1	45	45	52	45
14.9	148	146	149	150
14.9	292	292	290	290
19.7	385	386	386	393
38.1	463	470	470	466
Sum of (c)	418	425	418	421
No. of cracks (N)	5	5	5	5
Average (c) per side	84	85	84	84
Average (c)	84			

Crack Distances (mm) for Control Concrete Prisms Reinforcement Bar (B2)

Load (kN)	Control Concrete B2 Crack Distances (c) at Prism Faces (mm) A	B	C	D
36.6	45	60	55	53
15.1	149	152	162	160
23.4	248	282	279	285
18.9	375	380	388	385
Sum of (c)	330	320	333	332
No. of cracks (N)	4	4	4	4
Average (c) per side	83	80	83	83
Average (c)	82			

Crack Distances (mm) for Control Concrete Prisms Reinforcement Bar (B3)

Load (kN)	Control Concrete B3 Crack Distances (c) at Prism Faces (mm) A	B	C	D
38.4	88	85	88	88
19.1	119	123	125	124
36.2	235	240	220	230
15.6	290	288	285	287
23.2	398	400	393	395
Sum of (c)	310	315	305	307
No. of cracks (N)	5	5	5	5
Average (c) per side	62	63	61	61
Average (Δ c) All	62			

Crack Distances (mm) for 100% Recycled Concrete Prisms Reinforcement Bar (B1)

Load (kN)	100% Recycled Concrete B1 Crack Distances (c) at Prism Faces (mm)			
	A	B	C	D
40.1	68	52	65	67
20.6	202	200	198	200
24.2	305	305	304	302
43.1	384	383	374	385
Sum of (c)	316	331	309	318
No. of cracks (N)	4	4	4	4
Average (c) per side	79	83	77	80
Average (c)	80			

Crack Distances (mm) for 100% Recycled Concrete Prisms Reinforcement Bar (B2)

Load (kN)	100% Recycled Concrete B2 Crack Distances (c) at Prism Faces (mm)			
	A	B	C	D
41.2	140	133	132	140
19.2	228	226	224	226
17.9	330	335	330	330
45.1	413	426	415	410
Sum of (c)	273	293	283	270
No. of cracks (N)	4	4	4	4
Average (c) per side	68	73	71	68
Average (c)	70			

Crack Distances (mm) for 100% Recycled Concrete Prisms Reinforcement Bar (B3)

Load (kN)	100% Recycled Concrete B3 Crack Distances (c) at Prism Faces (mm)			
	A	B	C	D
43.1	38	40	45	40
19.3	165	157	160	163
43.0	358	360	362	368
Sum of (c)	320	320	317	328
No. of cracks (N)	3	3	3	3
Average (c) per side	107	107	106	109
Average (c)	107			

Crack Distances for Control Mortar Prisms Reinforcement Bar (B1)

Load (kN)	Control Mortar B1 Crack Distances (c) at Prism Faces (mm)			
	A	B	C	D
47.0	32	60	60	50
16.5	190	185	187	192
20.5	340	340	340	344
46.0	390	390	390	402
Sum of (c)	358	330	330	352
No. of cracks (N)	4	4	4	4
Average (c) per side	90	83	83	88
Average (c)	86			

Crack Distances (mm) for Control Mortar Prisms Reinforcement Bar (B2)

Load (kN)	Control Mortar B2 Crack Distances (c) at Prism Faces (mm)			
	A	B	C	D
35.0	55	50	40	55
14.2	230	230	220	220
16.4	284	280	275	280
33.5	450	455	455	455
Sum of (c)	395	405	415	400
No. of cracks (N)	4	4	4	4
Average (c) per side	99	101	104	100
Average (c)	101			

Crack Distances (mm) for Control Mortar Prisms Reinforcement Bar (B3)

Load (kN)	Control Mortar B3 Crack Distances (c) at Prism Faces (mm)			
	A	B	C	D
38.5	44	42	40	40
22.1	200	195	195	190
19.8	335	325	330	320
Sum of (c)	291	283	290	280
No. of cracks (N)	3	3	3	3
Average (c) per side	97	94	97	93
Average (c)	95			

Crack Distances (mm) for 100% Recycled Mortar Prisms Reinforcement Bar (B1)

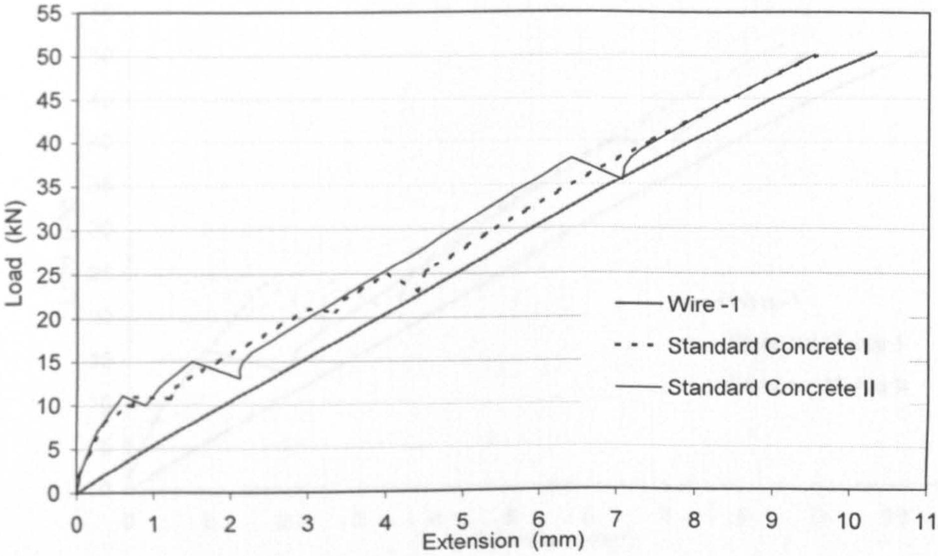
	100% Recycled Mortar B1			
	Crack Distances (c) at Prism Faces (mm)			
Load (kN)	A	B	C	D
40.0	32	30	45	45
13.7	222	219	219	220
21.3	370	375	375	380
40.0	470	475	475	-
Sum of (c)	438	445	430	335
No. of cracks (N)	4	4	4	3
Average (c) per side	110	111	108	112
Average (c)	110			

Crack Distances (mm) for 100% Recycled Mortar Prisms Reinforcement Bar (B2)

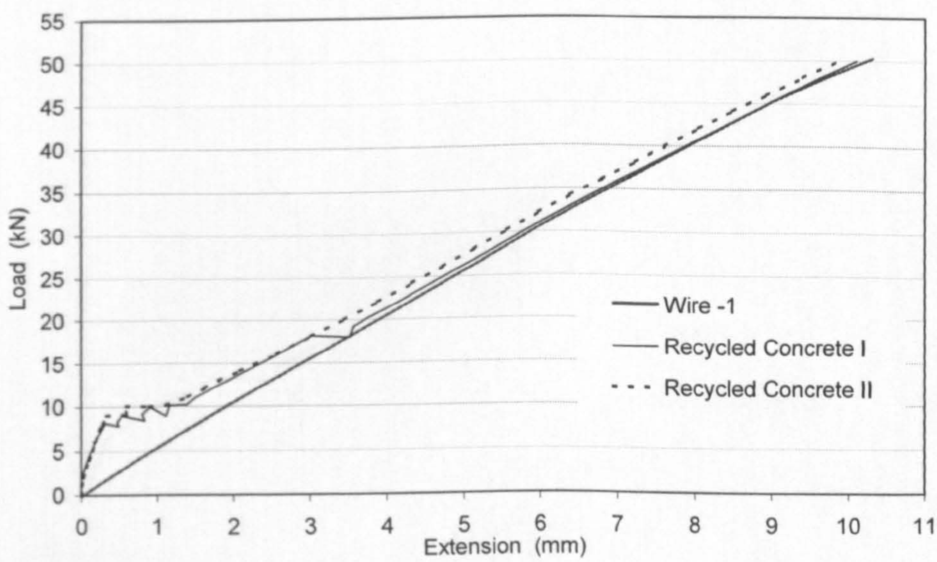
	100% Recycled Mortar B2			
	Crack Distances (c) at Prism Faces (mm)			
Load (kN)	A	B	C	D
37.0	50	50	45	-
15.1	224	227	223	225
41.0	470	450	450	-
Sum of (c)	420	400	405	0
No. of cracks (N)	3	3	3	1
Average (c) per side	140	133	135	0
Average (c)	136			

Crack Distances (mm) for 100% Recycled Mortar Prisms Reinforcement Bar (B3)

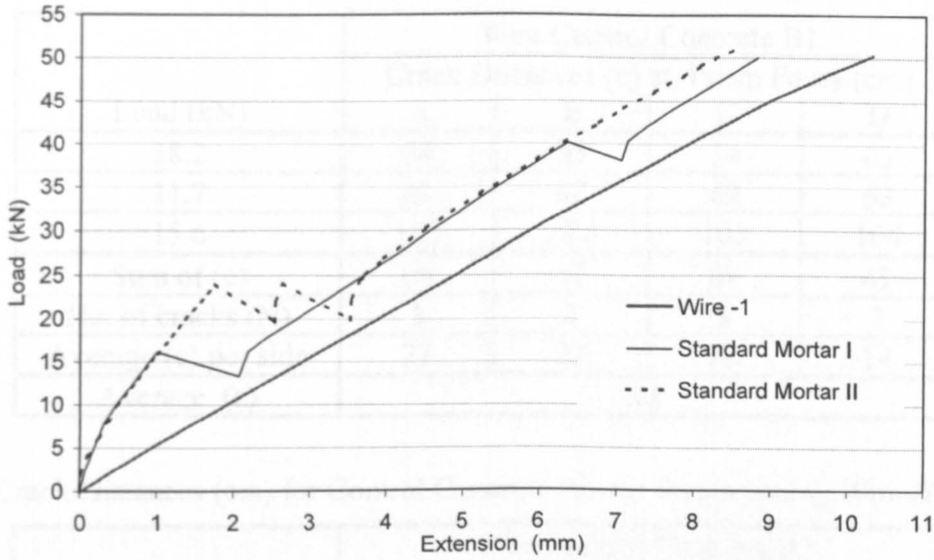
	100% Recycled Mortar B3			
	Crack Distances (c) at Prism Faces (mm)			
Load (kN)	A	B	C	D
21.5	188	184	184	184
14.8	300	302	304	300
Sum of (c)	112	118	120	116
No. of cracks (N)	2	2	2	2
Average (c) per side	56	59	60	58
Average (c)	58			



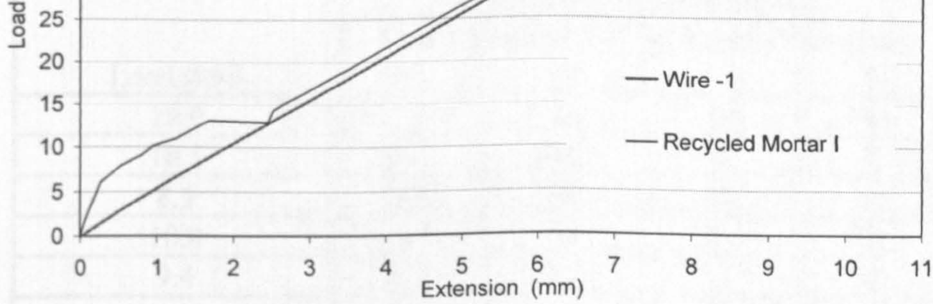
Tension versus elongation responses of standard concrete and pretensioning wire



Tension versus elongation responses of 100% RCA concrete and pretensioning wire



Tension versus elongation responses of standard mortar and pretensioning wire



Tension versus elongation responses of 100% RCFA mortar and pretensioning wire

Crack Distances (cm) for Control Concrete Prisms Pretensioning Wire (B1)

Load (kN)	Wire Control Concrete B1 Crack Distances (c) at Prism Faces (cm) A	B	C	D
38.2	34	35	34	63
11.7	63	62	62	63
15.6	103	102	103	104
Sum of (c)	69	67	69	41
No. of cracks (N)	3	3	3	3
Average (c) per side	23	22	23	14
Average (c)	21			

Crack Distances (cm) for Control Concrete Prisms Pretensioning Wire (B2)

Load (kN)	Wire Control Concrete B2 Crack Distances (c) at Prism Faces (cm) A	B	C	D
9.5	28	28	28	28
10.5	60	60	60	61
21.3	79	79	79	78
12.0	100	100	99	99
24.5	120	120	121	120
Sum of (c)	92	92	93	92
No. of cracks (N)	5	5	5	5
Average (c) per side	18	18	19	18
Average (c)	18			

Crack Distances (cm) for 100% RCA Concrete Prisms Pretensioning Wire (B1)

Load (kN)	Wire 100% RCA Concrete B1 Crack Distances (c) at Prism Faces (cm) A	B	C	D
19.0	35	30	29	30
10.5	53	51	51	52
8.5	68	68	68	68
10.0	94	93	94	95
9.4	121	121	121	121
Sum of (c)	86	91	92	91
No. of cracks (N)	5	5	5	5
Average (c) per side	17	18	18	18
Average (c)	18			

Crack Distances (cm) for 100% RCA Concrete Prisms Pretensioning Wire (B2)

Load (kN)	Wire 100% RCA Concrete B2 Crack Distances (c) at Prism Faces (cm) A	B	C	D
10.8	43	43	33	43
10.8	73	73	74	73
10.8	95	93	94	93
10.0	109	109	109	109
Sum of (c)	67	66	76	66
No. of cracks (N)	4	4	4	4
Average (c) per side	17	17	19	17
Average (c)	17			

Crack Distances (cm) for Control Mortar Prisms Pretensioning Wire (B1)

Load (kN)	Wire Control Mortar B1 Crack Distances (c) at Prism Faces (cm) A	B	C	D
16.0	43	43	43	43
40.5	67	66	66	67
15.7	92	92	91	92
Sum of (c)	49	49	49	50
No. of cracks (N)	3	3	3	3
Average (c) per side	16	16	16	17
Average (c)	16			

Crack Distances (cm) for Control Mortar Prisms Pretensioning Wire (B2)

Load (kN)	Wire Control Mortar B2 Crack Distances (c) at Prism Faces (cm) A	B	C	D
24.5	47	47	47	47
24.7	82	82	83	82
55.7	108	108	107	97
Sum of (c)	61	61	60	50
No. of cracks (N)	3	3	3	3
Average (c) per side	20	20	20	17
Average (c)	19			

Crack Distances (cm) for 100% RCFA Mortar Prisms Pretensioning Wire (B1)

Load (kN)	Wire 100% RCFA Mortar B1 Crack Distances (c) at Prism Faces (cm)			
	A	B	C	D
13.1	28	28	27	28
9.0	52	51	52	52
14.0	76	77	77	77
9.0	103	102	102	103
13.0	120	121	120	120
Sum of (c)	92	93	93	92
No. of cracks (N)	5	5	5	5
Average (c) per side	18	19	19	18
Average (c)	19			

Crack Distances (cm) for 100% RCFA Mortar Prisms Pretensioning Wire (B2)

Load (kN)	Wire 100% RCFA Mortar B2 Crack Distances (c) at Prism Faces (cm)			
	A	B	C	D
16.0	48	48	48	48
15.0	70	69	70	70
14.0	102	102	103	102
Sum of (c)	55	54	55	54
No. of cracks (N)	3	3	3	3
Average (c) per side	18	18	18	18
Average (c)	18			

Reinforcement Bar - at first crack stage
Force on the reinforcement Bar (P) vs. Tension force at the prism-beam (Ft)

Bar Force P (kN)	Stress (N/mm²)	Development Length (mm) (for 1st crack)	f_{cu} (N/mm²)	f_{ts} (N/mm²)	Beam tension Force Ft (kN)	P/Ft	f_b (N/mm²)
Standard concrete							
14.9	189.8	148.2	56.5	3.8	21.4	0.70	3.20
15.1	192.4	154.4	56.5	3.7	20.8	0.73	3.12
15.6	198.7	212	55.5	3.9	21.9	0.71	2.34
100% RCA Concrete							
20.6	262.4	200.4	33.0	2.8	15.5	1.33	3.27
17.9	228.0	169	34.0	2.9	16.0	1.12	3.37
19.3	245.9	162	32.0	2.8	15.8	1.23	3.79

Bar Force P (kN)	Stress (N/mm²)	Development Length (mm) (for 1st crack)	f_{cu} (N/mm²)	f_{ts} (N/mm²)	Beam tension Force Ft (kN)	P/Ft	f_b (N/mm²)
Standard Mortar							
16.5	210.2	188.8	45.0	3.9	21.9	0.75	2.78
14.2	180.9	226	46.5	4.0	22.2	0.64	2.00
19.8	252.2	171	45.5	3.8	21.4	0.93	3.69
100% RCFA Mortar							
13.7	174.5	220.4	32.0	2.4	13.5	1.02	1.98
15.1	192.4	224.6	33.5	2.8	15.8	0.96	2.14
14.8	188.5	185.6	30.5	2.7	14.9	0.99	2.54

AT FIRST CRACK	Average of all samples						
	Bar Force P (kN)	Stress (N/mm²)	Development Length (mm) (for 1st crack)	f_{ts} (N/mm²)	Beam tension Force Ft (kN)	P/Ft	f_b (N/mm²)
Standard Concrete	15.21	193.63	171.53	3.80	21.38	0.71	2.89
100% RCA Concrete	19.28	245.43	177.13	2.80	15.75	1.23	3.48
Standard Mortar	16.85	214.43	195.27	3.88	21.84	0.77	2.82
100% RCFA Mortar	14.55	185.13	210.20	2.62	14.72	0.99	2.22

Pre-stress wire - at first crack stage
Force on the pre-stress wire (P) vs. Tension force at the prism-beam (Ft)

Bar Force P (kN)	Stress (N/mm^2)	Development Length (mm) (for 1st crack)	f_{cu} (N/mm^2)	f_{ts} (N/mm^2)	Beam tension Force Ft (kN)	P/Ft	f_b (N/mm^2)
Standard concrete - wire							
11.7	304.0	624.0	56.5	3.8	21.4	0.5	0.9
9.5	247.0	280.0	56.5	3.7	20.8	0.5	1.5
100% RCA Concrete - wire							
8.5	220.8	679	33.0	2.8	15.5	0.55	0.57
8.0	208.5	408	34.0	2.9	16.0	0.50	0.89

Bar Force P (kN)	Stress (N/mm^2)	Development Length (mm) (for 1st crack)	f_{cu} (N/mm^2)	f_{ts} (N/mm^2)	Beam tension Force Ft (kN)	P/Ft	f_b (N/mm^2)
Standard Mortar - wire							
15.7	407.8	585.0	45.0	3.9	21.9	0.7	1.2
24.5	636.4	470.0	46.5	4.0	22.2	1.1	2.4
100% RCFA Mortar - wire							
9.0	234.0	475.4	32.0	2.4	13.5	0.67	0.86
14.0	363.6	478.0	33.5	2.8	15.8	0.89	1.33

AT FIRST CRACK	Average of all samples						
	Bar Force P (kN)	Stress (N/mm^2)	Development Length (mm) (for 1st crack)	f_{ts} (N/mm^2)	Beam tension Force Ft (kN)	P/Ft	f_b (N/mm^2)
Standard Concrete	10.6	275.5	452.0	3.8	21.1	0.5	1.2
100% RCA Concrete	8.26	214.65	543.50	2.80	15.75	0.53	0.73
Standard Mortar	20.1	522.1	692.5	3.9	22.1	0.9	1.6
100% RCFA Mortar	11.50	298.80	476.70	2.60	14.63	0.78	1.10

Appendix 4

1. X-beam unit Design.
2. Concrete Modulus of Elasticity
3. Average Strain Loss %
4. I_{cr} : Moment of inertia at cracking point
5. Cracking Moment & Ultimate Moment Failure

X-beam unite Design

Unite properties:
Length = 2000
Width = 200 mm
Depth = 215 mm
One core of 150 mm diameter

Concrete Data:

Compressive strength $f$cu = 40 - 55 N/mm^2
Ec = 32 kN/mm^2
Eci = 27 kN/mm^2
Wires: 3 @ 7 mm ; (area = 38.5 x 3 = 115.5 mm^2)
Bottom cover to wire = 25 mm (centre to 3 wire to bottom = 35)
e = 77.87
A concrete = 24676.4 mm^2 A total = 42355.0 mm^2 A circle =17678.6mm^2
Depth of concrete to core = 22.5 mm
Height of neutral axis = 112.87 mm ýt = (215-112.87) = 102.13

I = 136587661.4 mm^4
Z_b = 1210099.159 mm^3
Z_t = 1337430.993 mm^3
e = 77.87311489
A_{ps} = 115.5 mm^2
Es = 200 kN/mm^2
f_{pu} = 1670 N/mm^2

F= 192885 = 193 kN
F_i = 0.7x 192885 =
= 135019.5 N = 135 kN
f_{bc} = 14.16 N/mm^2
f_{ct} = -2.39 N/mm^2
f_{cc}=11.47 N/mm^2

Elastic loss = $f_{cc}\left(\frac{E_s}{E_{ci}}\right)$ = 84.93 N/mm^2 = 7.265 %
Creep loss = 1.8 x 7.265 = 13.077 %
Shrinkage loss = (300 x10^{-6}) x (200x10^3) = 60 N/mm^2 = 5.1 %
Relaxation Loss = 1.6 x 1.2 = 1.92 %
Total Losses = 27.395 % (Long term loss)
Total Losses around 28 days (estimated) = 18.5 %
Effective prestressing force after losses = Fi x (100-18.5) = 110 kN

After losses:

f_{bc} = 10.28 N/mm^2
f_{ct} = -1.74 N/mm^2

$\mathbf{M_{sr} = (f_{bc} + 0.45\sqrt{f_{cu}}) \times Z_b =}$	$\mathbf{M_{sr} = (f_{ct} + 0.33 f_{cu}) \times Z_t =}$
$\mathbf{M_{sr} = 17.8}$ **kNm**	$\mathbf{M_{sr} = 19.8}$ **kNm**

Standard Concrete Modulus of Elasticity

Natural 14 days					
Wt Air	Wt Water	Prism length	Frequency	Pulse time	Density
Kg	Kg	mm	Hz	microsec	kg/m3
12.093	7.043	500	3963	110.8	2395
12.108	7.041	500	3955	110.8	2390
12.037	7.005	500	3957	109.1	2392

Natural 14 days			
Ecq Erudite	Ecq Pundit	Average kN/mm2	
kN/mm2	kN/mm2	Ecq	Ec static
37.6	40.6	39.2	32.6
37.4	40.5		
37.5	41.9		

Natural 28 days					
Wt Air	Wt Water	Prism length	Frequency	Pulse time	Density
Kg	Kg	mm	Hz	microsec	kg/m3
12.101	7.002	500	4003	107.8	2373
12.140	7.049	500	3999	107.7	2385
12.440	7.001	500	4003	108.6	2287

Natural 28 days			
Ecq Erudite	Ecq Pundit	Average kN/mm2	
kN/mm2	kN/mm2	Ecq	Ec static
38.0	42.4	39.7	32.9
38.1	42.7		
36.6	40.2		

50%RCA Concrete Modulus of Elasticity

50%RCA 14 days					
Wt Air	Wt Water	Prism length	Frequency	Pulse time	Density
Kg	Kg	mm	Hz	microsec	kg/m3
11.520	6.478	500	3688	118.8	2285
11.630	6.517	500	3666	119.5	2275
11.611	6.558	500	3711	117.4	2298

50%RCA 14 days			
Ecq Erudite	Ecq Pundit	Average kN/mm2	
kN/mm2	kN/mm2	Ecq	Ec static
31.1	33.7	32.5	27.0
30.6	33.2		
31.6	34.7		

50%RCA 28 days					
Wt Air	Wt Water	Prism length	Frequency	Pulse time	Density
Kg	Kg	mm	Hz	microsec	kg/m3
11.530	6.488	500	3719	116.7	2287
11.612	6.566	500	3722	117.5	2301
11.620	6.568	500	3766	116.0	2300

50%RCA 28 days			
Ecq Erudite	Ecq Pundit	Average kN/mm2	
kN/mm2	kN/mm2	Ecq	Ec static
31.6	35.0	33.6	27.9
31.9	34.7		
32.6	35.6		

Average Strain Loss %

SG	NA	20%RCA	50% RCA	20% RCCA	50% RCCA
SG - 1	**12**	**14**	**16**	**11**	**15**
SG - 2	30	25	34	26	29
SG - 3	61	60	83	95	68

	Initial	Final	% loss
NA-SG1	4707	4101	13
NA-SG2	4654	3929	16
NA-SG3	4608	1540	67
2NA-SG1	4697	4172	11
2NA-SG2	4656	2613	44
2NA-SG3	4615	2047	56

20% RCA	Initial	Final	% loss
20RCA-SG1	4448	3891	13
20RCA-SG2	4304	3476	19
20RCA-SG3	4176	1702	59
20RCA2-SG1	4169	3509	16
20RCA2-SG2	4311	3005	30
20RCA2-SG3	4235	1627	62

20% RCCA	Initial	Final	% loss
20RCCA-SG1	4900	-	-
20RCCA-SG2	5000	-	-
20RCCA-SG3	5100	-	-
20RCCA2-SG1	4602	4114	11
20RCCA2-SG2	4545	3360	26
20RCCA2-SG3	4570	250	95

50% RCA	Initial	Final	% loss
50RCA-SG1	4867	4172	14
50RCA-SG2	4797	3157	34
50RCA-SG3	4745	250	95
50RCA2-SG1	4514	3711	18
50RCA2-SG2	4512	3025	33
50RCA2-SG3	4447	1275	71

50% RCCA	Initial	Final	% loss
50RCCA-SG1	4294	3707	14
50RCCA-SG2	4200	3593	14
50RCCA-SG3	4150	1620	61
50RCCA2-SG1	4358	3614	17
50RCCA2-SG2	4332	2465	43
50RCCA2-SG3	4346	1073	75

I_{cr} : Moment of inertia at cracking point

Unite properties:

Length = 2000
Width = 200 mm
Depth = 215 mm
One core of 150 mm diameter

Es = 200 kN/mm^2
Aps = 115.5 mm^2
fpu = 1670 N/mm^2

Concrete Data:

Compressive strength f_{cu} = 40 - 55 N/mm^2

E_c = 32 kN/mm^2

E_{ci} = 27 kN/mm^2

Wires: 3 @ 7 mm ; (area = 38.5 x 3 = 115.5 mm^2)
Bottom cover to wire = 25 mm (centre to 3 wire to bottom = 35)

$A_{concrete}$ = 24676.4 mm^2 A_{total} = 42355.0 mm^2 A_{circle} =17678.6mm^2

transforming wire to concrete= Aps

Aps = (Es / Ec) x Ast = (200/32)*115.5
Aps =722

1st moment of Inertia for remaining concrete (including tranfomed wires) =

1st Moment = [((200)(X^2))/2)] + [722 x 180]

Area =(200)(x) + 722

Xc = [1st moment] / [Area]

X = {[100 X^2] + [129960] / [(200)(X) + 722] }

0 = (100) (X^2) + 722 X - 129960

a = 100
b = 722
c = -129960

X= -39.8
X = 33

I_{cr} = [(b)(X^3)/3] + [Aps (d -X)^2]

I_{cr} = 1.79E+07

Cracking Moment & Ultimate Moment Failure

from BS 8110 (4-3-7)
Mu = fbp (Aps) (d-dn) Eq (51)

x/d = 0.207 using tabe 4-4 BS 8110
X = 0.207 x 180 =37.26 dn =18.63

Mu = 31.12585245kN.m

	Design Mom kN.m	Calc Mom with actual loss	Test Moment kN.m	Test Ult Failure kN.m	Calculated Mur
NA		20.28	20.3	28.5	
20% RCA		19.55	18.5	23.1	
50% RCA	**16.3**	20.10	18.0	23.5	**31.1**
20% RCCA		20.43	19.4	23.6	
50% RCCA		19.92	18.7	22.0	

BEFORE LOSSES	
fbc =	14.16046378
fcc =	11.46619687

mm4

Zb =	**1210099.159**	**1.210099159**
Zt =	**1337430.993**	

Msr = (ƒbc + 0.45 √Fcu) x Zb
[Msr / Zb] = fbc + ft
Msr with actual loss = Zb [fbc + ft]
fbc : stress after loss
ft : tensile strength (tests) chapter 5

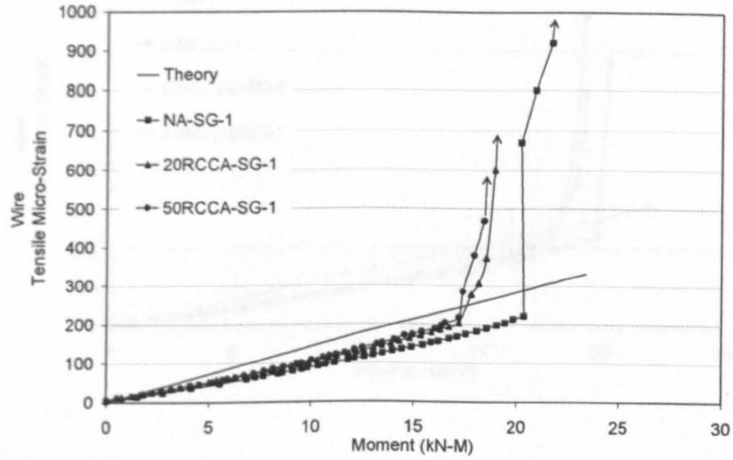
	ft Average	ft cone	ft jaw	ft impact
N.A	**4.3**	**4.3**	**4.3**	**4.3**
20% RCA	**4.0**	**4.0**	**3.8**	**4.2**
50% RCA	**4.7**	**4.6**	**4.9**	**4.7**
20% RCCA	**4.3**	**4.1**	**4.4**	**4.4**
50% RCCA	**4.4**	**4.8**	**4.0**	**4.5**

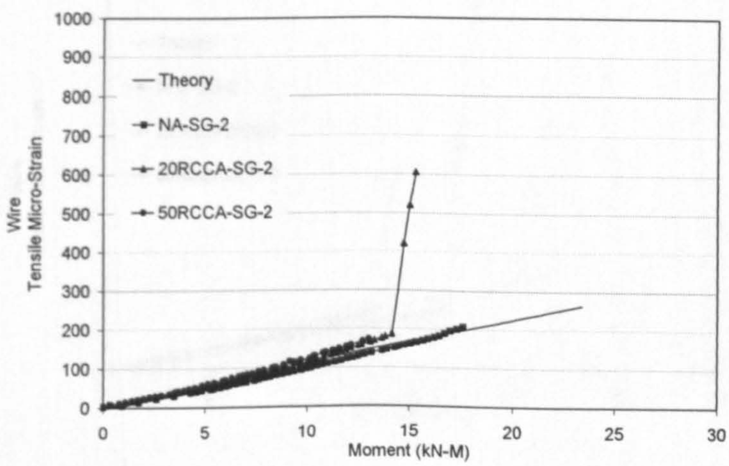
	SG - 1 Avg strain loss %	fbc after loss N/mm2 Strain after loss	ft	Calc Mom with actual loss
NA	12	12.5	4.3	20.28
20 RCA	14	12.2	4.0	19.55
50% RCA	16	11.9	4.7	20.10
20% RCCA	11	12.6	4.3	20.43
50% RCCA	15	12.0	4.4	19.92

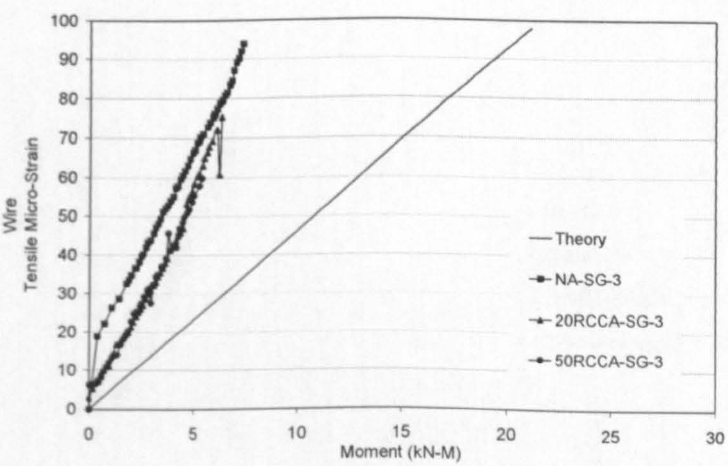
Appendix 5

1. Wire Strain for X-beam with RCCA (set 1)

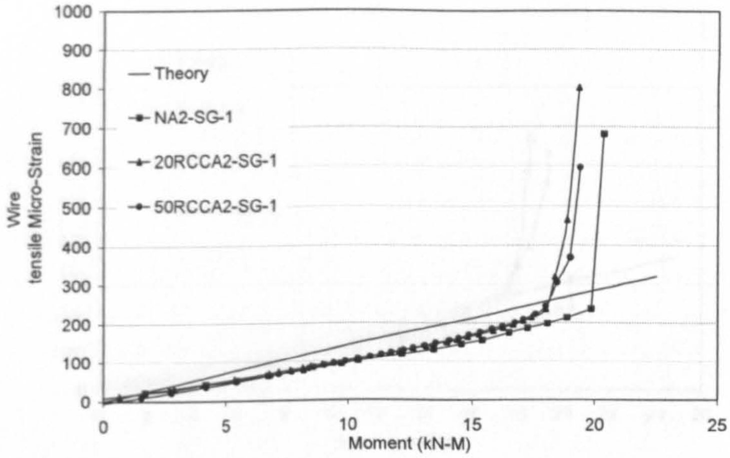
Wire Strain for X-beam with RCCA (set 1)

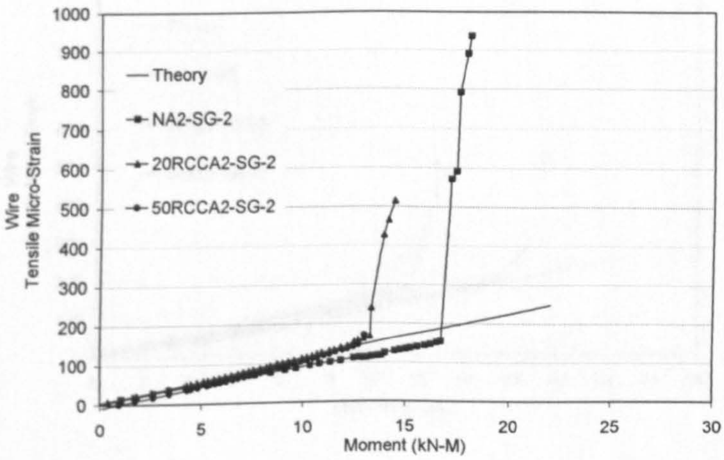


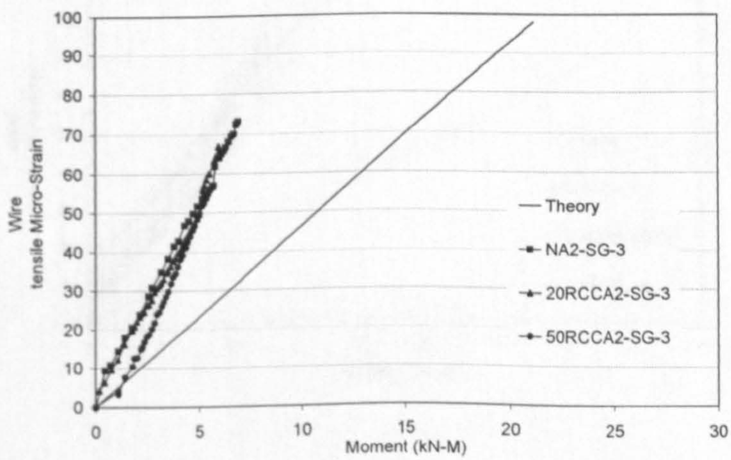




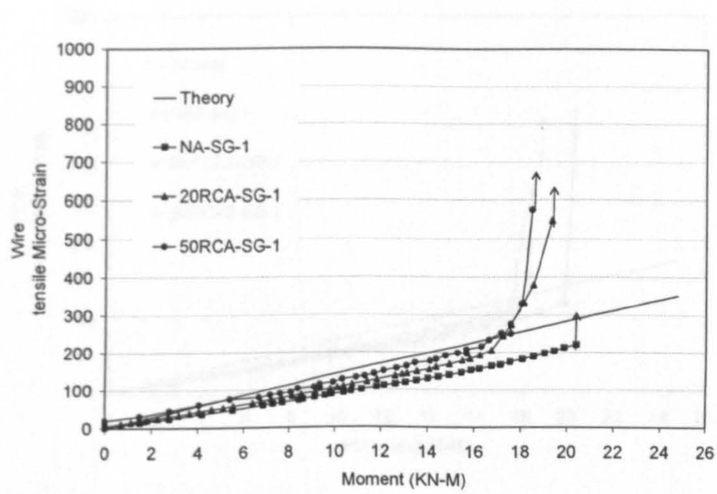
Wire Strain for X-beam with RCCA (set 2)

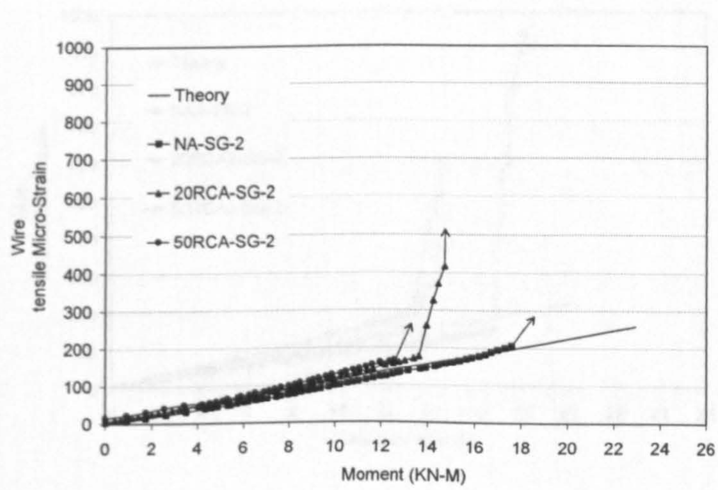


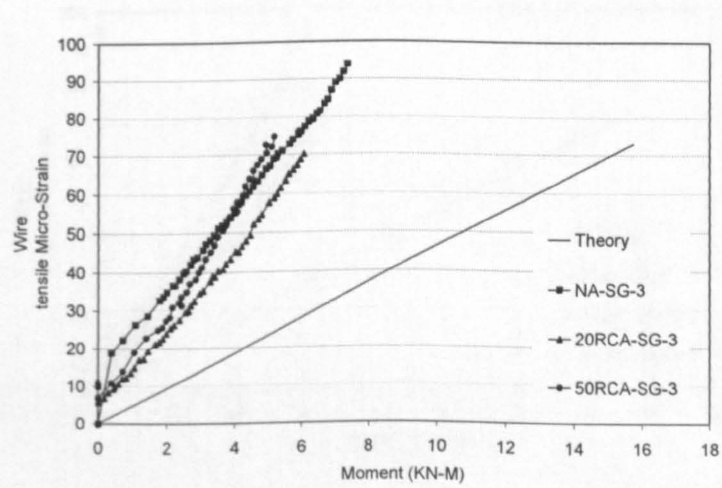




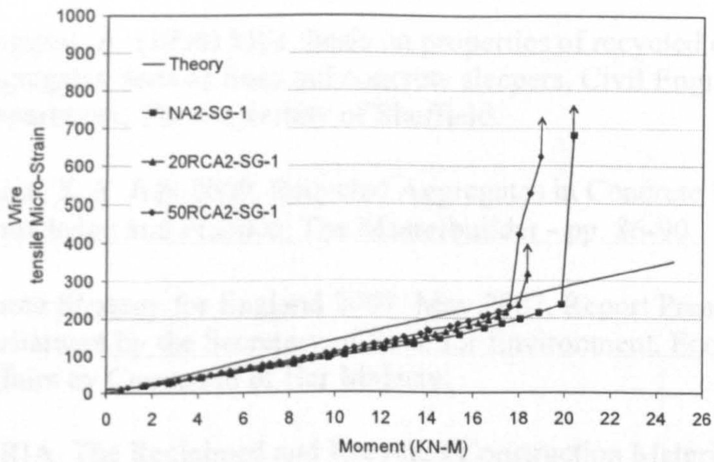
Wire Strain for X-beam with RCA (set 1)

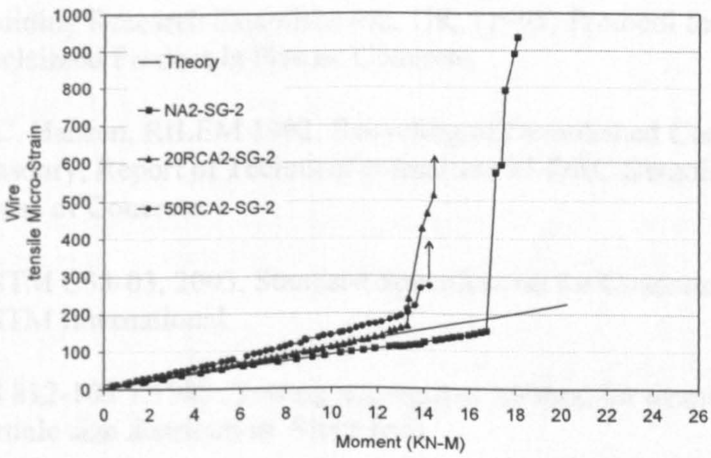


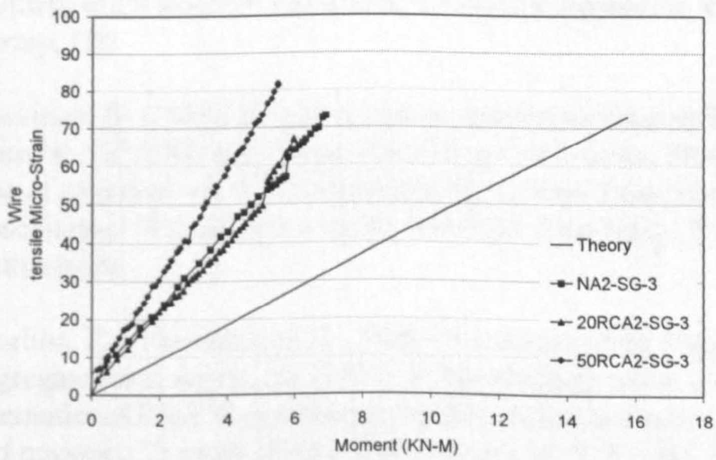




Wire Strain for X-beam with RCA (set 2)







References

1. Goggins, A. (1996) MSc thesis on properties of recycled concrete aggregates derived from rail concrete sleepers, Civil Engineering Department, The University of Sheffield.

2. Paine, K A. July 2009, Recycled Aggregates in Concrete Fundamentals, Knowledge and Practice, The Masterbuilder - pp. 86-90.

3. Waste Strategy for England 2007; May 2007, Report Presented to Parliament by the Secretary of State for Environment, Food and Rural Affairs by Command of Her Majesty.

4. CIRIA, The Reclaimed and Recycled Construction Materials Handbook, Part 4 Aggregates.

5. Building Research Establishment, UK, (1998) Protocol for the Use of Reclaimed Product in Precast Concrete.

6. T.C. Hansen, RILEM 1992, Recycling of Demolished Concrete and Masonry, Report of Technical committee 37-DRC Demolition and Reuse of Concrete.

7. ASTM C33-03, 2003, Standard Specification for Concrete Aggregates, ASTM International.

8. BS 812-103.1:1985: Testing aggregates: Method for determination of particle size distribution. Sieve tests

9. Mulheron, M., (1986) A preliminary study of recycled aggregates. The Institute of Demolition Engineers. 18 Station Approach, Virginia water, Surrey, UK.

10. Boesman, B. (1985), Crushing and separation techniques for demolition material. EDA/RILEM Demo-Recycling Conference, Proc. Vol. 2. Re-Use of concrete and Brick Materials, Rotterdam, European Demolition Association, Wassenaarseweg 80, 2596 CZ, Den Haag, The Netherlands.

11. Morlion, D., Venstermans, J., 1988, Demoltion of the Zandvliet lock as aggregate for concrete, pp. 709-718: Proceedings of the second international RILEM symposium on demolition and reuse of concrete and masonry. 2: reuse of demoltion waste (ed. Y. Kasai), Nihon Daigaku Kaikan, Tokyo, Japan, Chapman & Hall, London.

12. Hansen, T.C. and Narud, H. (1983) Strength of recycled concrete made with crushed concrete coarse aggregate. Concrete International, Design and Construction, 5, No. 1, pp.79-83.

13. Hedegaard, S. (1981), Recycling of concrete made of additives M.Sc. Thesis, Technical report 116/82. Building Material Laboratory, Technical University of Denmark Lyngby.

14. ASTM C457-98 10-Jul-1998, Standard Test Method for Microscopical Determination of Parameters of the Air-Void System in Hardened Concrete.

15. B.C.S.J. (1978) study on recycled aggregate and recycled aggregate concrete, Building contractors society of Japan, committee on disposal and reuse of concrete construction waste. Summary in concrete journal Japan, 16. No. 7, pp. 18-31.

16. Kobayasi and Kawano, 1988, Properties and usage of recycled aggregate concrete, pp 547-556, Proceedings of the second international RILEM symposium on demolition and reuse of concrete and masonry. 2: reuse of demolition waste (ed. Y. Kasai), Nihon Daigaku Kaikan, Tokyo, Chapman & Hall, London.

17. S. Nagatakia, A. Gokceb,*, T. Saekic, M. Hisadad (2004), Assessment of recycling process induced damage sensitivity of recycled concrete aggregates. Cement and Concrete Research, 34, p 965–971

18. Marta Sánchez de Juan , Pilar Alaejos Gutiérrez. (2009) Study on the influence of attached mortar content on the properties of recycled concrete aggregate. Construction and Building Materials 23 872–877

19. Hasaba, S., Kamwamura, M., Toriik, K. et al (1981), Drying Shrinkage and Durability of Concrete made of recycled concrete aggregates (translation of the Japanese Concrete Institution, 3, pp. 55-60.

20. Mulheron, M. and O'Mahony, M. (1987) Recycled aggregate. Properties and Performance. The Institute of Demolition Engineers, 18 Station Approach, Viginia Water Surrey.

21. Kreijger, P.C. (1983) Hergebruik van Bouw-en Sloopafval als Toeslagmateriaal in Beton. TH-Eindhoven, Adeling Bouwkunde, Rapport M83-1.

22. Hansen, T.C and Marga, M., 1988, Strength of recycled concrete made from coarse and fine recycled concrete aggregate, pp. 605-612: Proceedings of the second international RILEM symposium on demolition and reuse of concrete and masonry. 2: reuse of demoltion waste (ed. Y. Kasai), Nihon Daigaku Kaikan, Tokyo, Japan, Chapman & Hall, London..

23. ASTM C128-01e1, Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Fine Aggregate.

24. Vivian W.Y. Tam, X.F. Gao, C.M. Tam, C.H. Chan. (2008), New approach in measuring water absorption of recycled aggregates. Construction and Building Materials 22 364–369.

25. Merlet and Pimient, Mechanical and physico - chemical properties of concrete produced with coarse and fine recycled concrete aggregates, pp343-352, Demolition and reuse of concrete and masonry. Proceeding of the Third international RILEM Symposium. ED E K Lauritzen, Demesk Construction Engineers, Denmark. Odense, Denmark 1993. ISBN 041915820 0.

26. Kaga, H., Kasai, Y., Takeda K. and Kemi, T., 1988, Properties of Recycled aggregate from concrete, pp. 690-698: Proceedings of the second international RILEM symposium on demolition and reuse of concrete and masonry. 2: reuse of demoltion waste (ed. Y. Kasai), Nihon Daigaku Kaikan, Tokyo, Japan, Chapman & Hall, London.

27. Fergus, J.S., (1981), Laboratory Investigation and Mix Proportions for Utilising Recycled Portland Cement Concrete as Aggregate, Ibid, pp144-160.

28. BS 882, 1992 Specification for aggregates from natural sources for concrete. Replaced by BS EN 12620:2002

29. A.K. Padmini, K. Ramamurthy, M.S. Mathews. (2009), Influence of parent concrete on the properties of recycled aggregate concrete. Construction and Building Materials 23 829–836

30. Mukai, T., Kikuchi, M., and Koizumi, H. (1978) Fundamental Study on bond properties between recycled aggregate concrete and steel bars, Cement association of Japan 32nd review.

31. Ravindrarajah, R.S., and Tam T.C. (1985), Properties of concrete made with crushed concrete as coarse aggregate. Magazine of concrete research 37, No. 130.

32. Rasheeduzzafar and Khan, A. (1984) Recycled concrete – a source of new aggregate., Cement, concrete and aggregate, ASTM, 6, No. 1, pp.17-27.

33. C. S. Poon, S. C. Kou, L. Lam, (2007), Influence of recycled aggregate on slump and bleeding of fresh concrete, Materials and Structures 40:981–988

34. Nixon, P.J. (1978) Recycled concrete as an aggregate for concrete, a review RILEM TC-37-DRC, Materials and structures (RILEM) 65, pp.371-378.

35. Wainwright, P .J., Trevorrow, A., Wang, Y., YU, Y., (1993) Modifying the performance of concrete made with coarse and fine recycled concrete aggregates , pp319-329, Demolition and reuse of concrete and masonry. Proceeding of the Third international RILEM Symposium. ED E K Lauritzen, Demesk Construction Engineers, Denmark. Odense, Denmark, ISBN 0419158200.

36. Yoda, K., Yoshikane, T., Nakashima, Y. and Soshiroda, T., (1988), Recycled cement and recycled concrete in Japan, pp 527-536: Proceedings of the second international RILEM symposium on demolition and reuse of concrete and masonry. (ed. Y. Kasai), Nihon Daigaku Kaikan, Tokyo, Japan, Chapman & Hall, London.

37. Kawai , T., Watanabe, M. and Nagataki, S., (1988) Preplace aggregate concrete made from demolished concrete aggregates pp. 680-689: Proceedings of the second international RILEM symposium on demolition and reuse of concrete and masonry. 2: reuse of demolition waste (ed. Y. Kasai), Nihon Daigaku Kaikan, Tokyo, Japan, Chapman & Hall, London.

38. Coquillat, G. (1982), Recycled de materiaux de demolition dans la confection de Beton. CEBTP – Service d'Etude des Materaux Unite: Technologies des Beton (in French). Marche No.80-61-248 saint Remy les Chevreuse.

39. Gerardu J.J.A and Hendriks, C.F. (1985) Recycling of road pavement materials in the Netherlands. Rijkswaterstaat communications No. 38, the Hague.

40. Malhotra , V.M. (1978) Use of Recycled Concrete as a new aggregate, Proceedings of Symposium on Energy and Resource Conservation in the Cement and Concrete Industry CANMET, report no. 76-8 Ottawa.

41. Karaa, T., (1986) evaluation techniques des possibilities d'emplois des dechets dan la construction - recherché experimentale appliqué au cas de beton fabrique partier de granulats de betons recycles. These de doctorate de Universite Paris CSTB 4 Avenue du Recteur Poincare 75782 Paris Cedex 16 France.

42. Ikeda , T., Yamane, S. and Sakamoto, A., (1988), Strengths of concrete containing recycled aggregate, pp. 585-594: Proceedings of the second international RILEM symposium on demolition and reuse of concrete and masonry. 2: reuse of demolition waste (ed. Y. Kasai), Nihon Daigaku Kaikan, Tokyo, Japan, Chapman & Hall, London.

43. Kawamura, M. and Toriik, K. et al. (1981) drying shrinkage and durability of concrete made of recycling concrete made with aggregate obtained from demolished pavement. Journal of the Society of Materials Science, Japan, 32, No. 353.

44. G.V. Guineaa, K. El-Sayedb, C.G. Roccoc, M. Elicesa, J. Planasa, (2002). The effect of the bond between the matrix and the aggregates on the cracking mechanism and fracture parameters of concrete; Cement and Concrete Research 32, pg1961–1970

45. R. Collins Final Report March 1999, Reclaimed product in precast concrete – DETR Partners in innovation contract cc1528.

46. Jianzhuang Xiao a,*, H. Falkner b, (2007), Bond behavior between recycled aggregate concrete and steel rebars; Construction and Building Materials 21,pg 395–401.

47. H. B. Choi and K. I. Kang. (2008), Bond behaviour of deformed bars embedded in RAC, Magazine of Concrete Research, 60, No. 6, August, 399–410

48. Buck A.D. (1977) Recycled concrete as a source of aggregate, ACI Journal, pp. 212-219.

49. A L A Faaij, H S Pieterson, J de Vries, September 2002, Performance of concrete with recycled aggregate, Sustainable concrete construction, Proceeding of the international conference, Dundee university Editor R K Dhir, T D Dyer, J E Halliday.

50. T Fujii, (1988), Strength and drying shrinkage behaviour of concrete using concrete crushed aggregate. Proceedings of the second international RILEM symposium on demolition and reuse of concrete and masonry.ed. Y. Kasai, Nihon Daigaku Kaikan, Tokyo, Japan, November 1988, Chapman & Hall, London.

51. BS 6089:1981 Guide to assessment of concrete strength in existing structures

52. Mellor, S. H., (1990). An introduction to crushing and screening, Institute of Quarrying,.

53. G.T. BROWN, (1963), Principle and practice of crushing and screening.

54. Boesman, B., (1985) Crushing and separating techniques for demolition material. EDA/Rilem Demo-recycling conference. Re-Use of Concrete and Brick Materials, European Demotion Association, Den Haag, The Netherlands. Proc. Vol. 2.

55. D C Teychenne. (1992). Use of Crushed Rock Aggregates in Concrete, Department of The Environment, Building Research Establishment; BR18. ISBN 0 851250505 Published 1978, Reprinted.

56. BS 812. Part 105-1, (1989). Testing aggregates. Methods for determination of particle shape. Flakiness index

57. BS 812, Part1, (1975). Testing aggregates. Methods for determination of particle size and shape

58. Kaplan. M. F.. Flexural and compressive strength of concrete as affected by the properties of coarse aggregate, J Amer. Concr. Inst., 55, 1959. pp 1193-208.

59. Neville. Adam M. (1995) Properties of Concrete, Harlow, Longman, , Fourth Edition

60. BS 812, Part 2, 1995. Testing aggregates. Methods for determination of density

61. BS 812, Part 111. 1990. Testing aggregates. Methods for determination of ten per cent fines value (TFV).

62. BS 812, Part 110, 1990, Testing aggregates. Methods for determination of aggregate crushing value (AIV)

63. Design of normal concrete mixes BRE, published by Department of the Environment, 1988. ISBN 085125222

64. BS 1881, Part 125, 1986, Testing concrete. Methods for mixing and sampling fresh concrete in the laboratory.

65. L. Evangelista. J. de Brito, (November 2004). "Criteria for the Use of Fine Recycled Concrete Aggregates in Concrete Production", conference on the use of recycled aggregate in building and structures, Barcelona

66. BS 1881, Part 102, 1991, Testing concrete. Method for determination of slump.

67. BS 1881. Part 103. 1991. Testing concrete. Method for determination of compacting factor.

68. G. H. Tattersall, (1991), Workability and quality control, E & FN Spon, first edition.

69. BS 1881. Part 116. 1983. Testing concrete. Method for determination of compressive strength of concrete cubes.

70. Yoda, K., Yoshikane, T., Nakashima, Y. and Soshiroda, T., (1988), Recycled cement and recycled concrete in Japan, pp 527-536: Proceedings of the second international RILEM symposium on demolition and reuse of concrete and masonry. (ed. Y. Kasai), Nihon Daigaku Kaikan, Tokyo, Japan, Chapman & Hall, London,.

71. BS 1881, Part 118, 1983, Testing concrete, Method for determination of flexural strength.

72. Kawamura, M. and Toriik, K. et al. (1981) drying shrinkage and durability of concrete made of recycling concrete made with aggregate obtained from demolished pavement. Journal of the Society of Materials Science, Japan, 32, No. 353.

73. Malhotra, V.M. (1978) Use of Recycled Concrete as a new aggregate, Proceedings of Symposium on Energy and Resource Conservation in the Cement and Concrete Industry CANMET, report no. 76-8 Ottawa.

74. Ikeda , T., Yamane, S. and Sakamoto, A., (1988), Strengths of concrete containing recycled aggregate, pp. 585-594: Proceedings of the second international RILEM symposium on demolition and reuse of concrete and masonry. 2: reuse of demolition waste (ed. Y. Kasai), Nihon Daigaku Kaikan, Tokyo, Japan, Chapman & Hall, London.

75. BS 8110, Part 1, 1997, Structural use of concrete, Code of practice for design and construction

76. BS, 1881, Part 117, 1983, Testing concrete. Method for determination of tensile splitting strength

77. Coquillat, G. (1982), Recycled de materiaux de demolition dans la confection de Beton. CEBTP – Service d'Etude des Materaux Unite, Technologies des Beton (in French), Marche No.80-61-248 saint Remy les Chevreuse.

78. BS 1881, Part 114. 1983, Testing concrete. Methods for determination of density of hardened concrete.

79. K.K. Sideris *, P. Manita, K. Sideris, (2004), School of Engineering, University of Thrace, Greece, Estimation of ultimate modulus of elasticity and Poisson ratio of normal concrete, Cement & Concrete Composites 623–631

80. S.-C. Kou and C.-S. Poon, Hong Kong, (2008), Mechanical properties of 5-year-old concrete prepared with recycled aggregates obtained from three different sources, Magazine of Concrete Research, 60, No. 1, February, 57–64

81. Akesson, M. (1993) Fracture Mechanics Analysis of the Transmission Zone in Prestressed Hollow Core Slabs Publication 93.5 Chambers University of Technology, Goteborg, Sweden.

82. Den Uijl, J. A. (1985) Bond properties of strands in connection with transmission zone cracks. Concrete Precasting Plant & Technology, Vol 51, No. 1.

83. Federation International de la Precontrainte (FIP) 1986 Recommendations. Design of Multi Storey Precast Concrete Structures, Thomas Telford, London.

84. The international federation for structural concrete (FIB), Volume 1, Issue 3, September 2000.

85. Pajari, M (1988) VTT Load carrying capacity of prestressed extruded hollow core slabs Nordic concrete research.

86. Lorentsen M., 1995 Steel Fiber Concrete for Structural Element, US-Sweden joint seminar, 3-5 June 1985, Stockholm, Sweden.

87. Kevin A. Paine, "Steel Fibre Reinforced Concrete For Prestressed Hollow Core Slabs", Ph.D Thesis, The University of Nottingham, September 1998.

88. BS 1881: 1983, Testing concrete. Method of normal curing of test specimens (20°C method).

89. Gustavson R., (March 2004), "Experimental studies of the bond response of three-wire strands and some influencing parameters", Materials and Structures / Matdriaux et Constructions, Vol. 37, pp 96-106

90. Homayon H Abrishami and Denis Mitchell, November-December 1996, "Influence of splitting Cracks on Tension Stiffeneing", ACI structural journal, V. 93, No. 6.

91. Associate Brasilerira de Normas Tecnicas (1980) "Ensaio de compressao em corpos-de-prova cilindricos de concerto 2, NBR 5739, Rio Dejeneiro, Brazil.

92. Jianzhuang Xiao a, H. Falkner b, (2007), Bond behavior between recycled aggregate concrete and steel rebars; Construction and Building Materials 21,pg 395–401.

93. BS 5896 (1981), Specification for high tensile steel wire and strand for the prestressing of concrete.

94. Elliott, Kim S, (2002), Precast concrete structures, Butterworth-Heinemann, ISBN 0750650842.